

PB-227 345

TEMPORARY DETENTION OF STORM AND
COMBINED SEWAGE IN NATURAL UNDERGROUND
FORMATIONS

Richard A. Hudak, et al

South St. Paul, Minnesota

Prepared for:

Environmental Protection Agency

May 1973

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PB 227 345

1	Accession Number	2	Subject Field & Group
W		SELECTED WATER RESOURCES ABSTRACTS INPUT TRANSACTION FORM	

5	Organization
City of South St. Paul, Minnesota	

6	Title
TEMPORARY DETENTION OF STORM AND COMBINED SEWAGE IN NATURAL UNDERGROUND FORMATIONS	

10	Author(s)	16	Project Designation
Hudak, Richard A. McMahon, Thomas J.		EPA, WQO Contract No. 11030DSL	
		21	Note

22	Citation
Environmental Protection Agency Report Number EPA-R2-73-242, May 1973.	

23	Descriptors (Starred First)
Combined Sewage, Underground Detention	

25	Identifiers (Starred First)
* Detention of Combined Storm and Sanitary Sewage in Underground Formations	

27	Abstract
The object of this study was to demonstrate the feasibility of temporarily detaining storm and combined sewage in natural underground formations. Five sites were selected for subsurface geological and geophysical investigation for the purpose of determining which site possessed subsurface conditions most suitable for storing and retrieving storm and combined sewage. The geophysical work required six resistivity soundings as well as resistivity survey involving five traverses. Based on this work, three sites were selected for test boring. A four-inch (4") test boring was made at each of these three sites. Two of the sites were too shallow for later demonstration of the technique. The third site was selected for the demonstration.	

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May 1973

TEMPORARY DETENTION OF STORM AND COMBINED SEWAGE
IN NATURAL UNDERGROUND FORMATIONS

By

City of St. Paul, Minnesota

Project 11030 DSL
Program Element 1B2034

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OFFICE OF RESEARCH AND MONITORING
U.S. ENVIRONMENTAL PROTECTION AGENCY
WASHINGTON, D.C. 20460

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ABSTRACT

The object of this study was to demonstrate the feasibility of temporarily detaining storm and combined sewage in natural underground formations and was to proceed in three phases.

Five sites were selected for subsurface geological and geophysical investigation for the purpose of determining which site possessed subsurface conditions most suitable for storing and retrieving storm and combined sewage.

The geophysical work required six resistivity soundings as well as resistivity survey involving five traverses. Based on this work, three sites were selected for test boring. A four-inch (4") test boring was made at each of these three sites. Two of the sites were too shallow for later demonstration of the technique. The third site was selected for the test pumping of Phase I. Because of the small underground storage available, the City of South St. Paul elected not to continue into Phases II and III.

Included in the scope of work for Phase I was an investigation, analysis, and discussion of methods of solids separation which might be used for storm and combined sewage in Phase II and III, prior to injection of the effluent underground.

Alternate proposals, including surface detention and treatment, and total separation of storm and sanitary sewers, were also considered.

This report was submitted in fulfillment of Project No. 11030 DSL, under the partial sponsorship of the Environmental Protection Agency.

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SECTION I

CONCLUSIONS

The storage of storm or combined sewage in underground formations requires geologic formations that possess sufficient physical capacity to permit injection of the sewage at a rate that will sufficiently match the incoming flow rate. Optimization of injection and inflow rates is imperative, if the need to provide large surface detention basins is to be avoided - particularly in urban areas.

Based on tests taken at five selected sites along the City river-front, the most suitable location permitted a maximum injection rate of 1000 GPM, or 1,400,000 gallons-per-day.

A rainfall equivalent to the test areas' five-year-storm frequency would result in some 70-million gallons of runoff, or a peak rate of over 2,500,000 GPM. Thus, the obvious necessity of providing storage basins to minimize the number and size of injection systems.

In addition, the studies point to a need to remove all solids larger than 5-microns. To accomplish this, sedimentation basins, chemical flocculation, and the use of sand filters or fine screens would be required.

The potential for contamination of underground water supply sources must also be carefully evaluated where such methods are considered.

It was determined that aforementioned facility requirements were not economically feasible in view of other methods available. In view of the need for sedimentation, flocculation, and filters or screens - it appears that, with few additions, the effluent could be discharged directly to the river.

Because of the findings, it was decided not to proceed with Phases II and III, the objectives of which are described in the introduction.

SECTION II

RECOMMENDATIONS

In order to demonstrate feasibility of the underground detention method of storm water, it is recommended that the Office of Research and Monitoring consider a new demonstration grant where a suitable underground formation with sufficient surface ponding area for solids separation and sufficient underground input capacity is available.

SECTION III

INTRODUCTION

The City of South St. Paul, Minnesota is a part of the seven county metropolitan area of the Twin Cities. It is located on the right bank of the Mississippi River adjacent to the southerly portion of St. Paul. It is bounded on the west by West St. Paul and on the south by Inver Grove Heights.

The City has a population of about 24,000 but due to the presence of large livestock and meat packing industries, the population equivalent for sewage treatment purposes was approximately 800,000 at the time of study, now decreased to 400,000 because of the closing down of a major meat packing industry. The sewage disposal plant at the time of the study was owned by the City but is presently owned by the Metropolitan Sewer Board and is in the process of being phased out and will act as a pretreatment facility for the packing industry waste. All domestic waste water and pretreated packing waste water will be pumped to the Pigs Eye Island treatment plant by a 48" force main.

The City is situated principally along the Mississippi River at the foot of and parallel to a line of bluffs which rise sharply from the valley on the west side of the river. During periods of high intensity rainfall, storm water runoff is high and is conveyed to the City system of combined storm and sanitary sewers. The Sewage Disposal Plant is bypassed during these periods and storm and sanitary sewage is discharged into the Mississippi River without treatment.

A study of the combined sewer problem in South St. Paul was undertaken and completed and data regarding the study has been provided in the Appendix of this report. As of one of the possible solutions to this problem and in order to demonstrate and report on the feasibility thereof, the City Council accepted, on July 22, 1968, an offer of a Federal Grant by the Federal Water Pollution Control Administration, predecessors to the EPA and hereafter referred in this report as EPA, for a demonstration project with temporary detention of storm and combined sewage in natural underground formations. The project was originally designated as Demonstration Project No. WPRD249-C1 and was subsequently changed to Project No. 11030DSL. Under an agreement of 9 September, 1968, the City of South St. Paul engaged MacKichan and Madsen to undertake the work. At the request of the EPA, the project was divided into three phases. Phase I consisted of selecting a site with a suitable natural underground formation into which storm water could be injected during Phase II and combined storm and sanitary sewage during Phase III. This report is concerned with the results of investigation and analysis on Phase I only of the project.

The objective of Phase I, II, and III of the Demonstration Project was to demonstrate and report on the feasibility of using natural, permeable underground formations in South St. Paul for temporary detention of both storm water and combined sewage during periods of peak runoff when the wastes must be diverted into the Mississippi River without treatment. This method could have possible national applications in areas with similar conditions. It was considered as one method of solving the combined sewer problem in the City of South St. Paul and to assist in the abatement of pollution in the Mississippi River.

This report is limited to a discussion of the results of the basic investigation called for in Phase I; however, the scope of work for all three phases is described hereafter.

Phase I has consisted of basic investigations to include geophysical, geological, and hydrological analysis of underground formations in South St. Paul, in order to select a site into which, in later phases, clear water, storm water, and combined sewage could be pumped, monitored and recovered for demonstration of the technique of temporary detention in permeable underground formations. Sufficient geophysical surveys, test drilling, and test pumping were undertaken to search for permeable formations suitable for developing the technique and subsequent phases. Phase I also included an investigation analysis and recommendations for a method of solid separation of storm and combined sewage to be tested in Phases II and III.

Phase II was to consist of demonstration of the system at the location selected in Phase I, in order to evaluate the technique under varied inflow conditions. In this phase, City water was to be introduced into the permeable formations by a combination of infiltration basins and gravity or purger wells within the retention pool. Direction and rate of water movement would be established by quantitative pumping test procedures. These procedures are established methods of evaluating movement of liquids in permeable formations. Observation piezometer wells would be used to monitor the changes in water quality as the liquid introduced underground moves through the formation. After testing with City water and after separation of solids by methods recommended in Phase I, storm water was to be introduced into the underground system. The storm water would be monitored, analyzed and finally recovered by pumping.

Phase III was to consist of demonstrating the feasibility for temporary underground storage of combinations of storm water and sanitary sewage. The problem of economically separating the solids from the combined waters would be further investigated and evaluated. The remaining effluent would then be diverted into the underground system, monitored and recovered. A final report was to be prepared which would present the results of the investigation, testing and evaluation undertaken in all three phases, along with recommendations for further application of the technique in South St. Paul and nationally.

It should be emphasized at this point that the basic purpose of Phase I was to locate a site in South St. Paul with an underground formation considered to be geologically and hydrologically suitable for actual demonstration of the technique and subsequent phases. The applicability of the temporary detention method in the City of South St. Paul and other parts of the nation could only be determined after demonstration of feasibility for storm water in Phase II and for combined sewage in Phase III.

Phase I of the project was carried out under the general direction of the EPA and under the direct supervision of the City of South St. Paul by their former City Engineer, Donald R. Peterson.

In execution of the work, the consultants, MacKichan and Madsen, retained the services of Eugene A. Hickok and Associates, consulting hydrologists of Wayzata, Minnesota, on the geophysical, geologic, and hydrologic aspects of the work. A contractor was also engaged to undertake test borings and drilling of the test well and observation wells.

To investigate, analyze, and make recommendations for methods to separate the solids from both storm and sanitary sewage, as was called for in Phases I and II, the services of Henningson, Durham & Richardson, Inc. Engineers, Architects, and Planners, Omaha, Nebraska, were retained to report on this matter. The findings of Eugene A. Hickok & Associates, and Henningson, Durham & Richardson have been presented in the context of this report.

SECTION IV

FIELD INVESTIGATIONS AND TESTS

In order to assess the potential locations for underground storage in South St. Paul, the initial step involved a study of maps and geologic literature, along with consultation with the City Engineer and Superintendent of Sewage Disposal Plant at South St. Paul. This was followed by field investigations, using electrical resistivity methods to determine bedrock depths.

Based on geologic and geophysical indications, the most favorable sites were tested by drilling. Plate 1 is a location map showing all five sites. Plates 2 through 6 are detailed maps showing the actual location of the electrical resistivity soundings and the test holes.

Preliminary planning limited the general field investigation to the strip of land between Concord Street and the Mississippi River. This area combines potential storage with the use of gravity flow and relatively uninhabited surroundings. The geophysical method chosen as being the most advantageous for the area was electrical resistivity. Geological reconnaissance indicate that, within this area, the bedrock consists of St. Peter Sandstone which is terraced in most places by stream action. Most of this terraced area within the existing valley was subsequently buried by more recent glacial and alluvial materials.

The target of all efforts was to locate permeable lenses within these recent sediments, preferably underlain by impervious beds and with enough surface area to allow for possible site development.

Electrical resistivity soundings were conducted at five sites in an attempt to determine the thickness of recent sediments overlying bedrock.

The basic operation in this geophysical method is the measurement of natural electrical resistivity in the ground. The various earth materials have differing degrees of resistance to an induced electric current, depending on the presence and nature of contained moisture. The moisture in a clay formation moves extremely slow, and the formation has a high content of salts dissolved from the clay, making it a good electrolyte and thus reducing its electrical resistivity. Sand and gravel permit relatively rapid movement of any contained water, and they form natural filters, which results in fresh water, with attendant high resistivity. Dry sand and gravel would have a high resistivity, and bedrock usually shows a definite high reading.

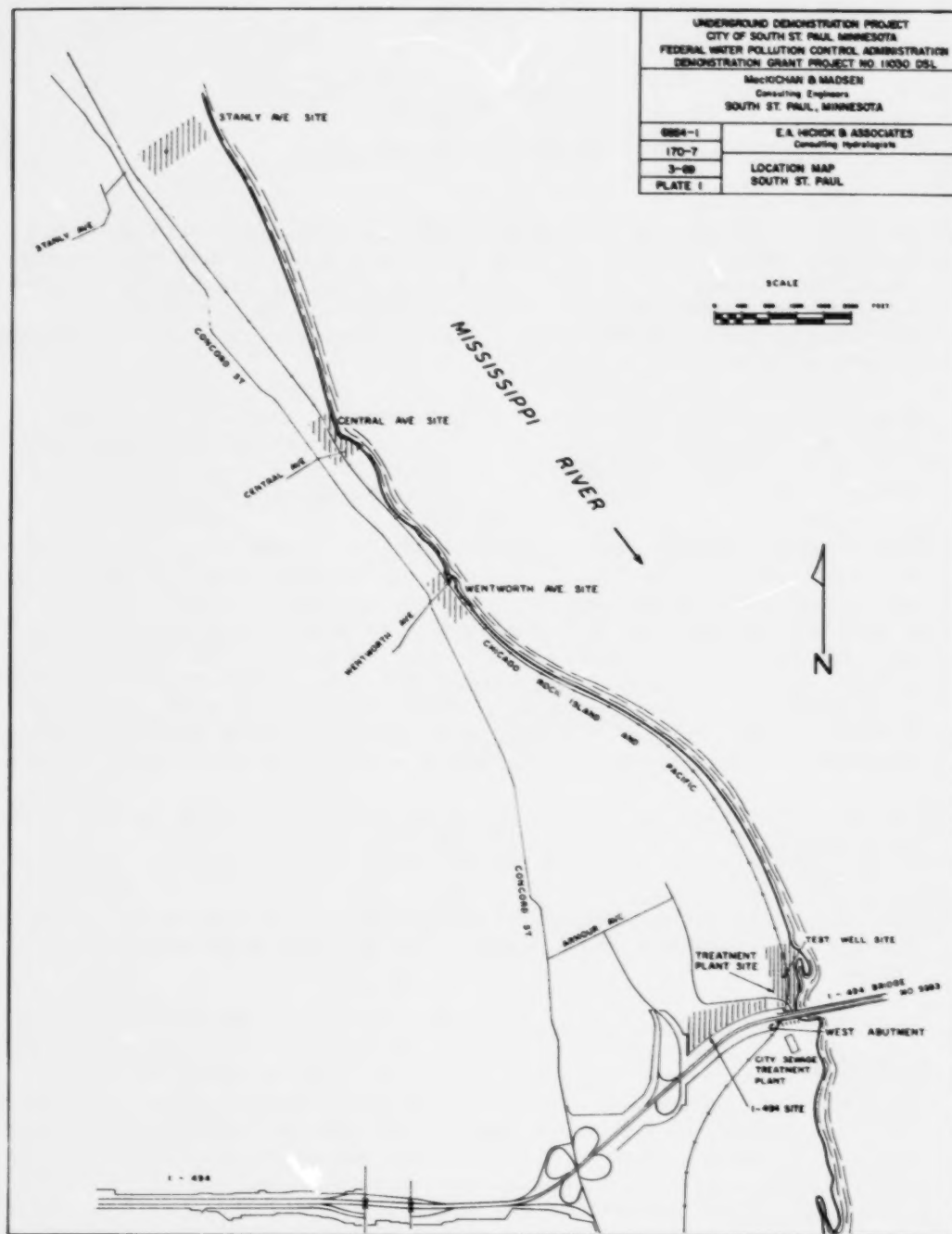


Figure 1.

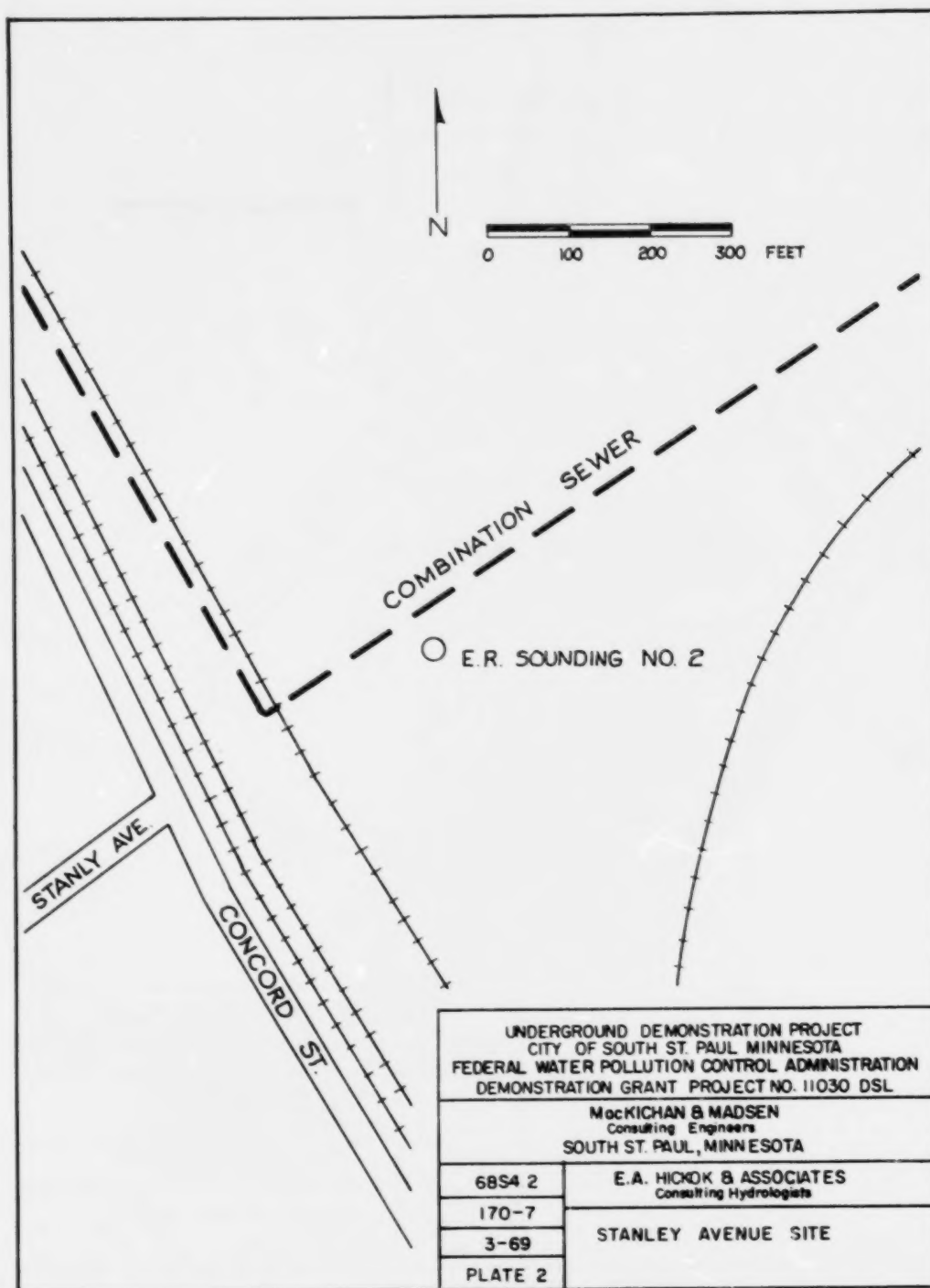


Figure 2.

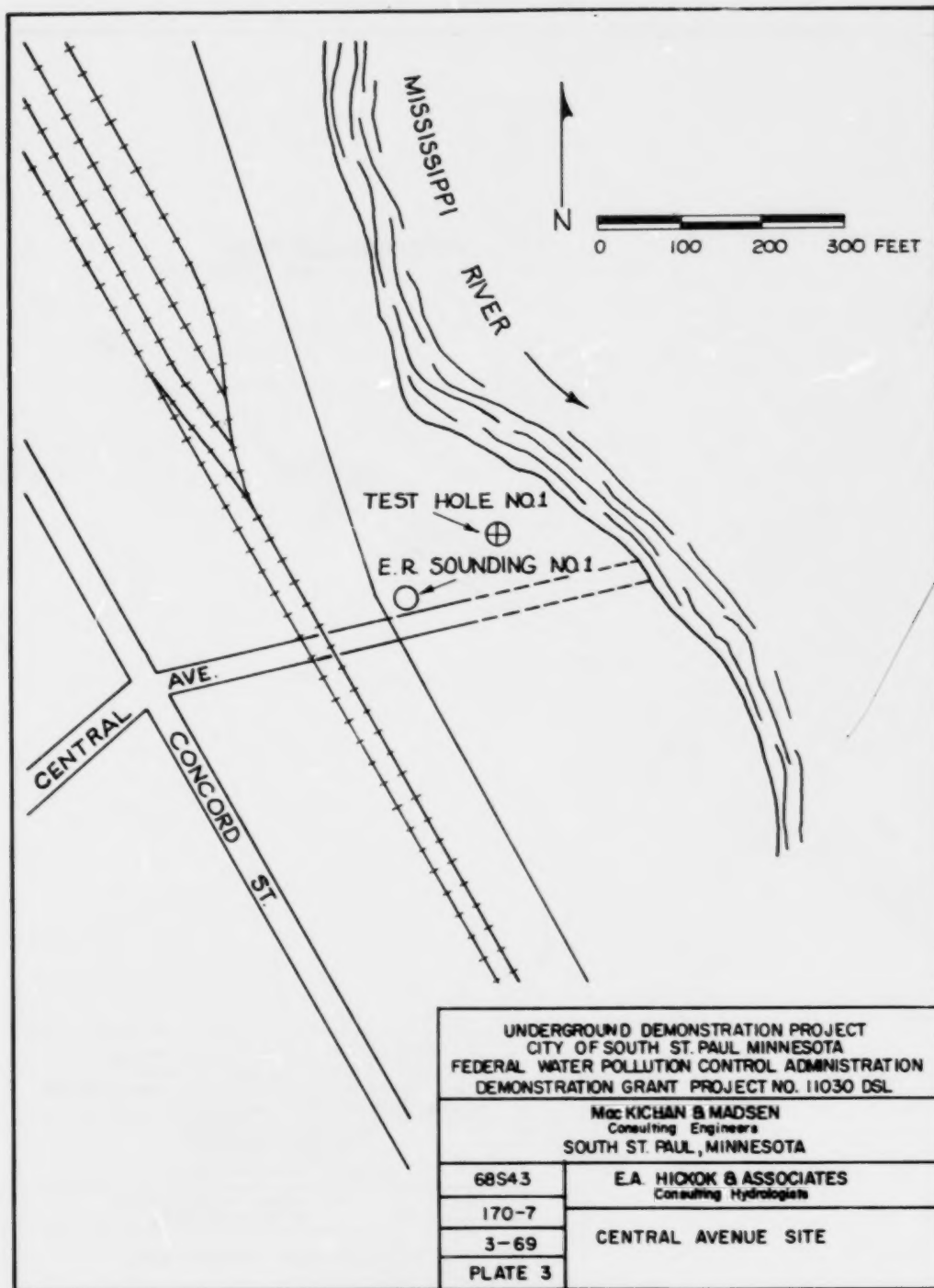


Figure 3.

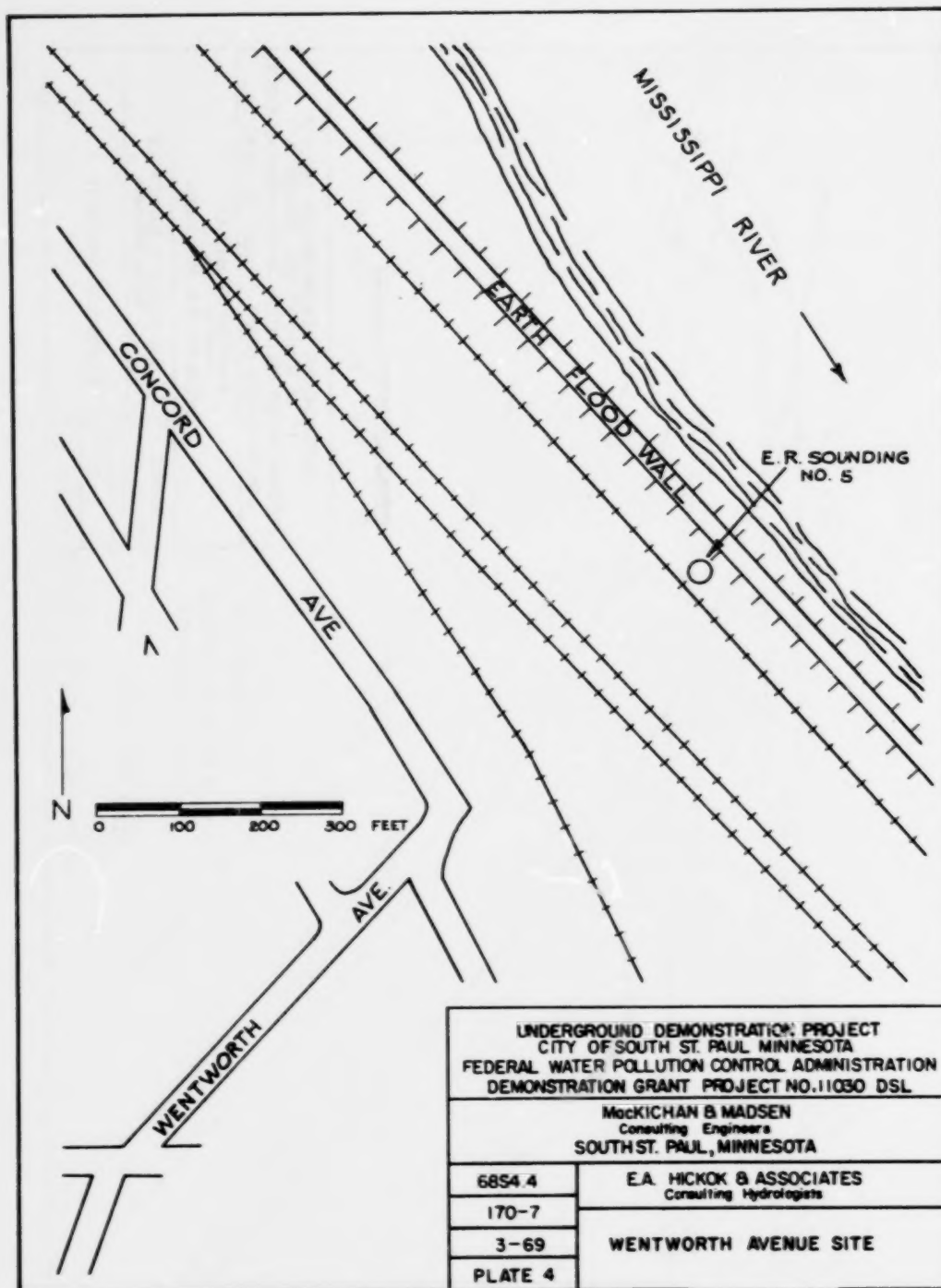


Figure 4.

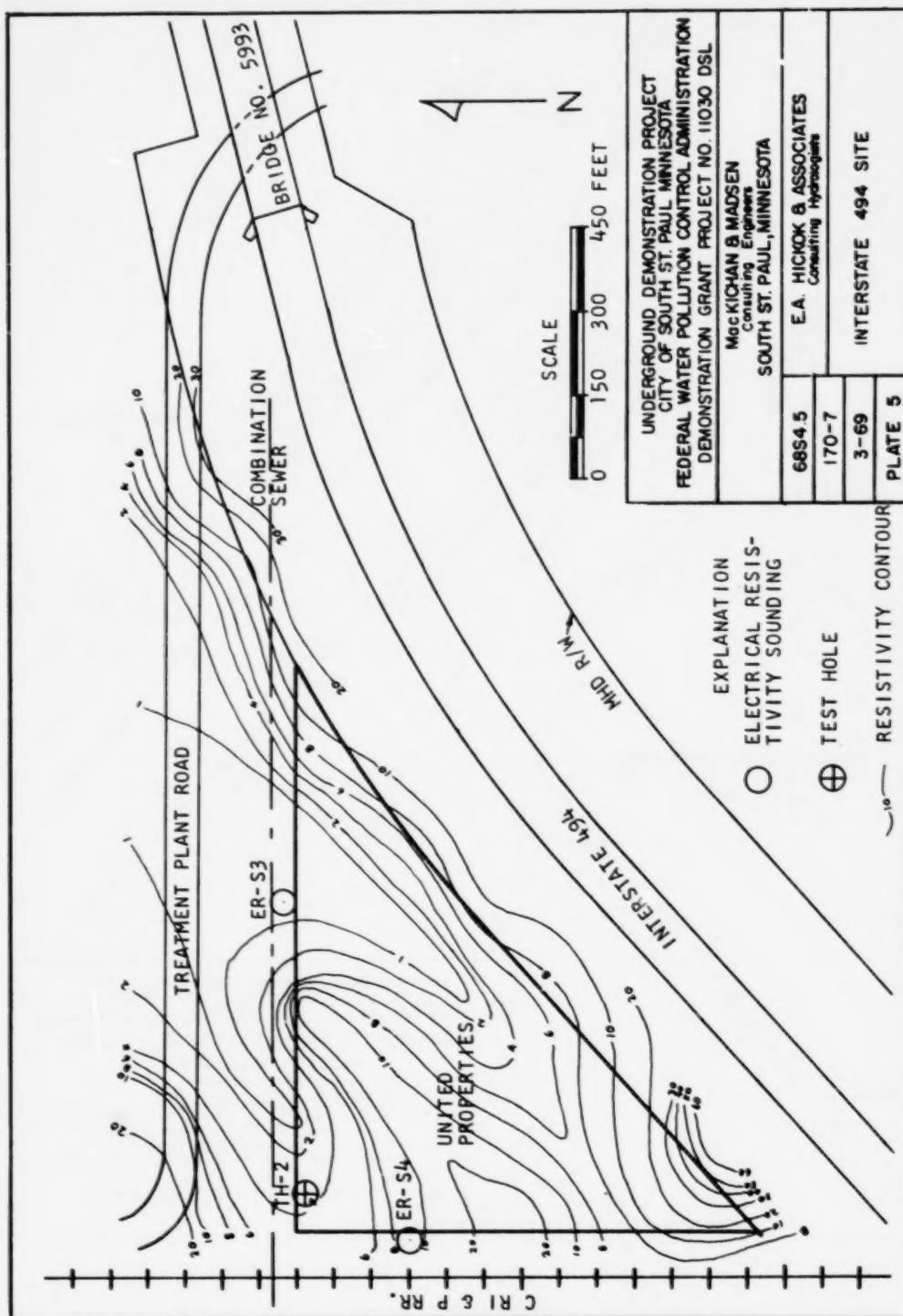


Figure 5.

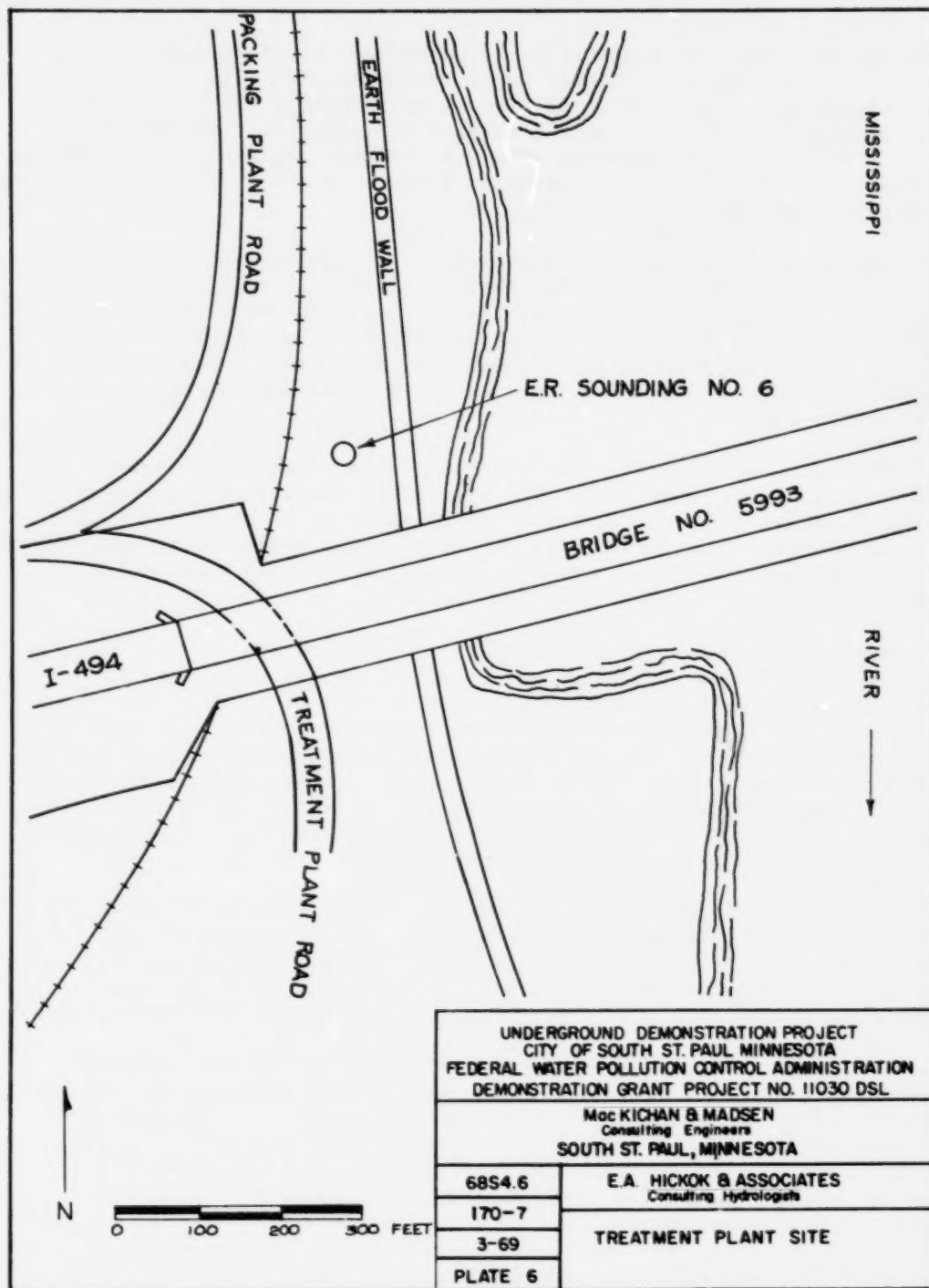


Figure 6.

In making an electrical resistivity sounding, electrode configurations are adjusted to read successively deeper measurements. The results of these soundings are plotted on graphs shown in Plates 7 thru 12. At the Interstate 494 site, a lateral electrical resistivity survey was made, in which several traverses were run, taking continuous measurements. The results of this survey are shown on Plate 5 in the form of apparent resistivity contours.

With completion of the geologic and geophysical investigation, results were analyzed and sites chosen for test drilling. It is usually desirable to correlate electrical resistivity data with known local subsurface conditions, such as drill hole information, as means of increasing the accuracy of interpretation. In this case, there was no opportunity to do so and, in effect, the first test hole has to provide this information.

The first hole, TH-1, at the Central Avenue site, found limestone bedrock at a depth of 30 feet. The electrical resistivity sounding of this location indicated a bedrock depth range of 25 to 40 feet. The site was considered unsuitable because of a direct hydraulic connection between the limestone bedrock and the overlying alluvium that would allow contamination of the underlying aquifers.

The second hole, TH-2, was located at the Interstate 494 site. For space and drill access, the hole was located near the west property line. The electrical resistivity sounding at this site, ER3, ER4, and ER6, showed an increase in bedrock depth from west to east. At the west line the sounding data was somewhat ambiguous, indicating possible bedrock depths of 20 feet and 55 feet. Sounding ER-S3 showed a more definite depth indication of about 50 feet. Drill hole TH-2 encountered limestone bedrock at 30 feet. This location was considered unsuitable for the same reason as the Central Avenue site.

Test holes were not taken at the Stanley Avenue and Wentworth sites. Poor accessibility and limited working space was one reason for not putting down a test hole. The electrical resistivity soundings at both sites indicated bedrock formations at depths of 25 to 40 feet, thus making it unsuitable for the same reason as the Central Avenue site.

Minnesota Highway Department borings at the west end of the Interstate 494 Bridge, as seen on Plate 13, show that alluvial deposits thicken to 100 feet going east from TH-2 to the bridge abutment. On the basis of this information, another test site was chosen East of TH-2 and north of the Interstate 494 Bridge. The final location was determined by the need of 150 lineal feet that avoided roads, railroad track, low ground and power line construction. The first hole at this site, TH-3, was drilled to a 125 foot depth, all in alluvium. Two more holes, TH-4 and TH-5, were drilled, each to a 125 foot total depth. All of these holes were subsequently re-numbered. The southernmost, an 8-inch diameter hole to be test-pumped, was designated Test Well No. 1, TW-1.

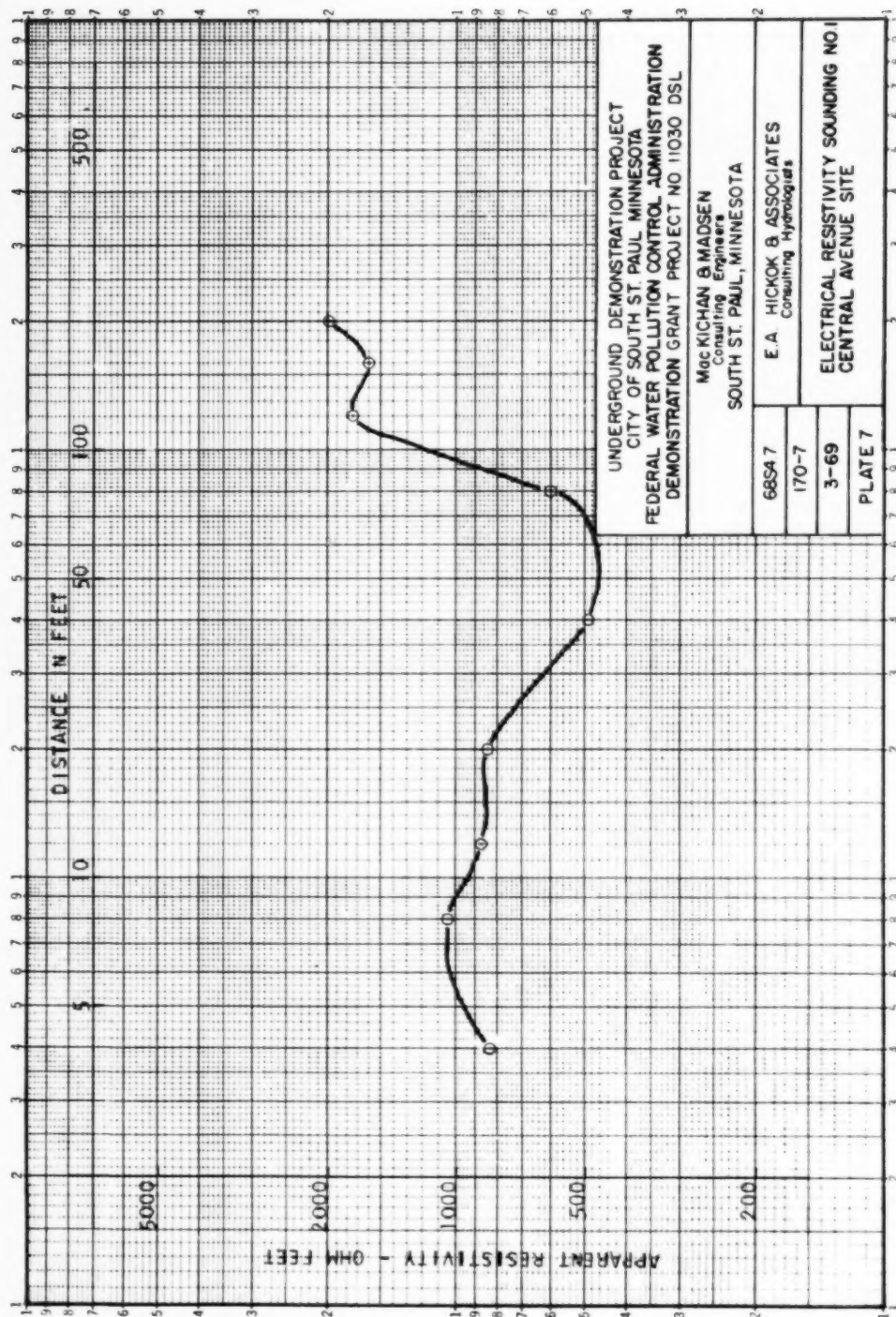


Figure 7.

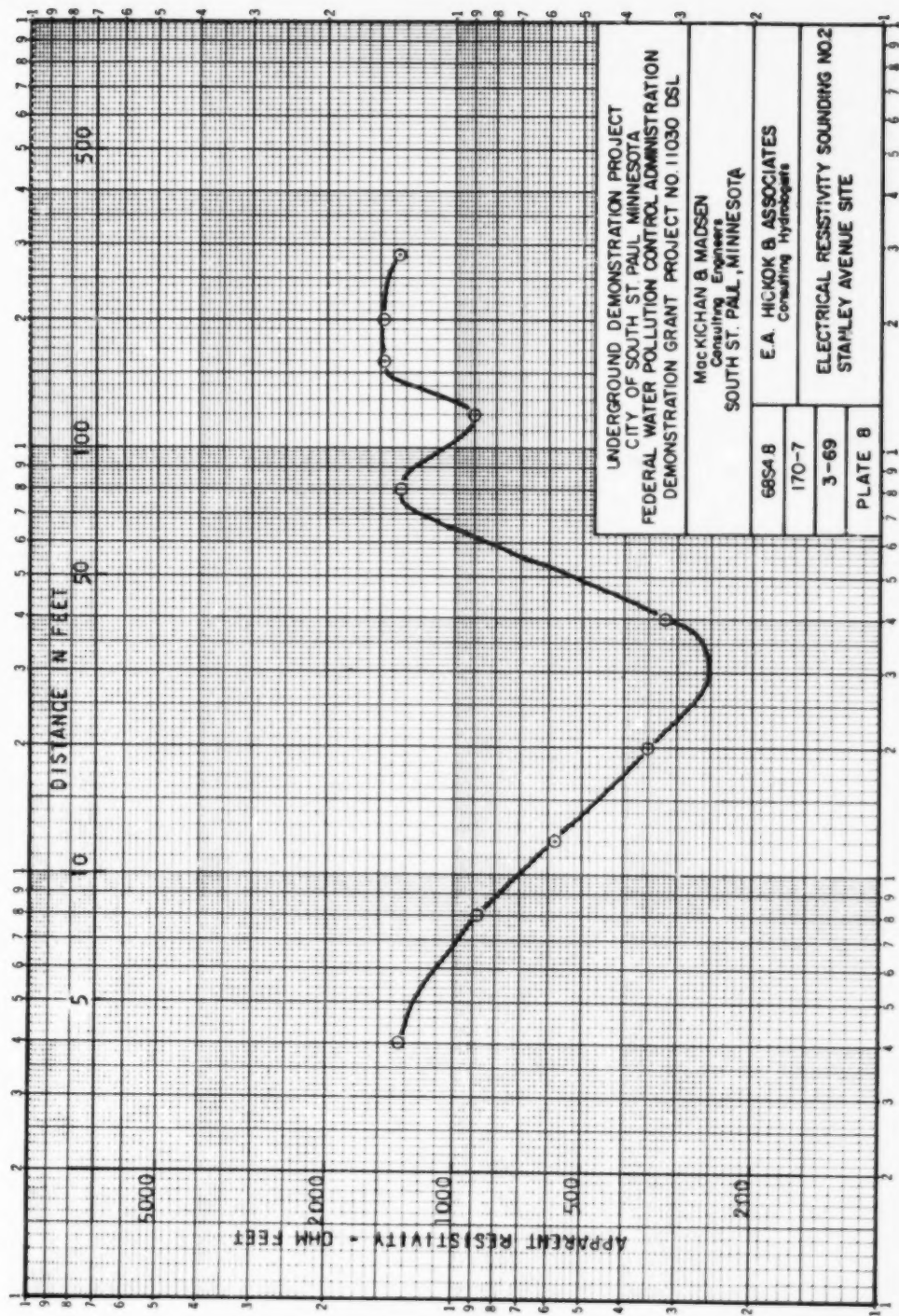


Figure 8.

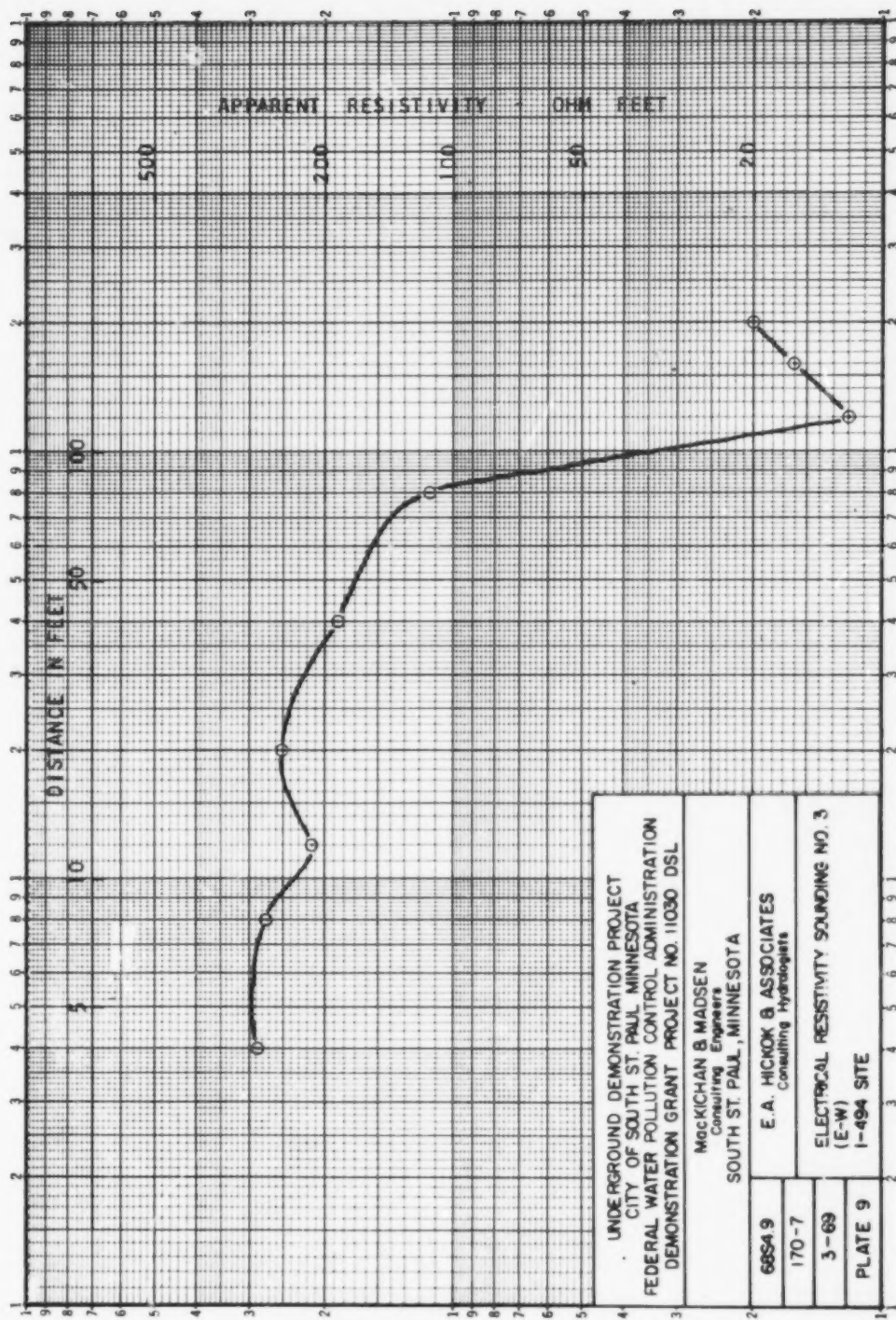


Figure 9.

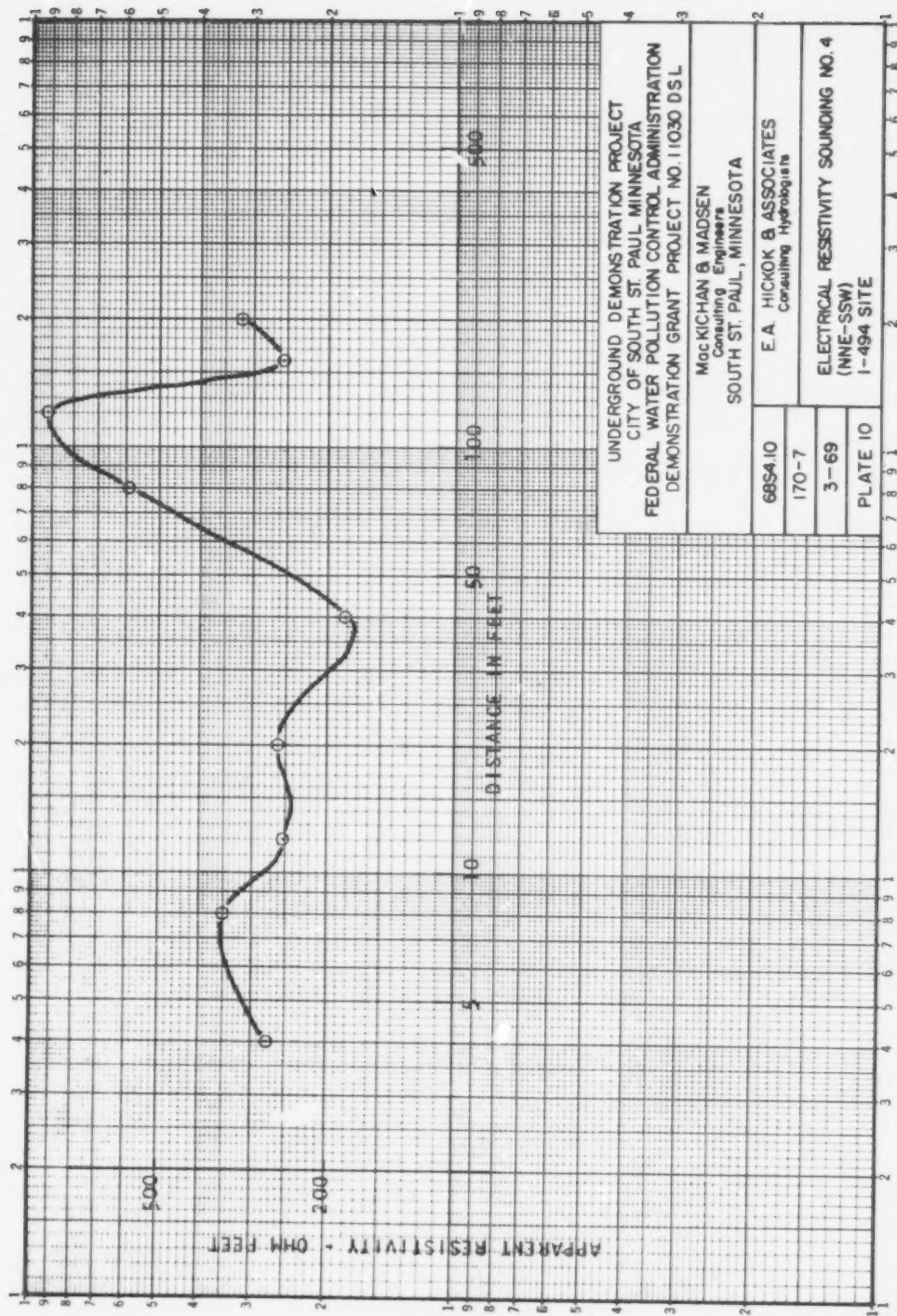


Figure 10.

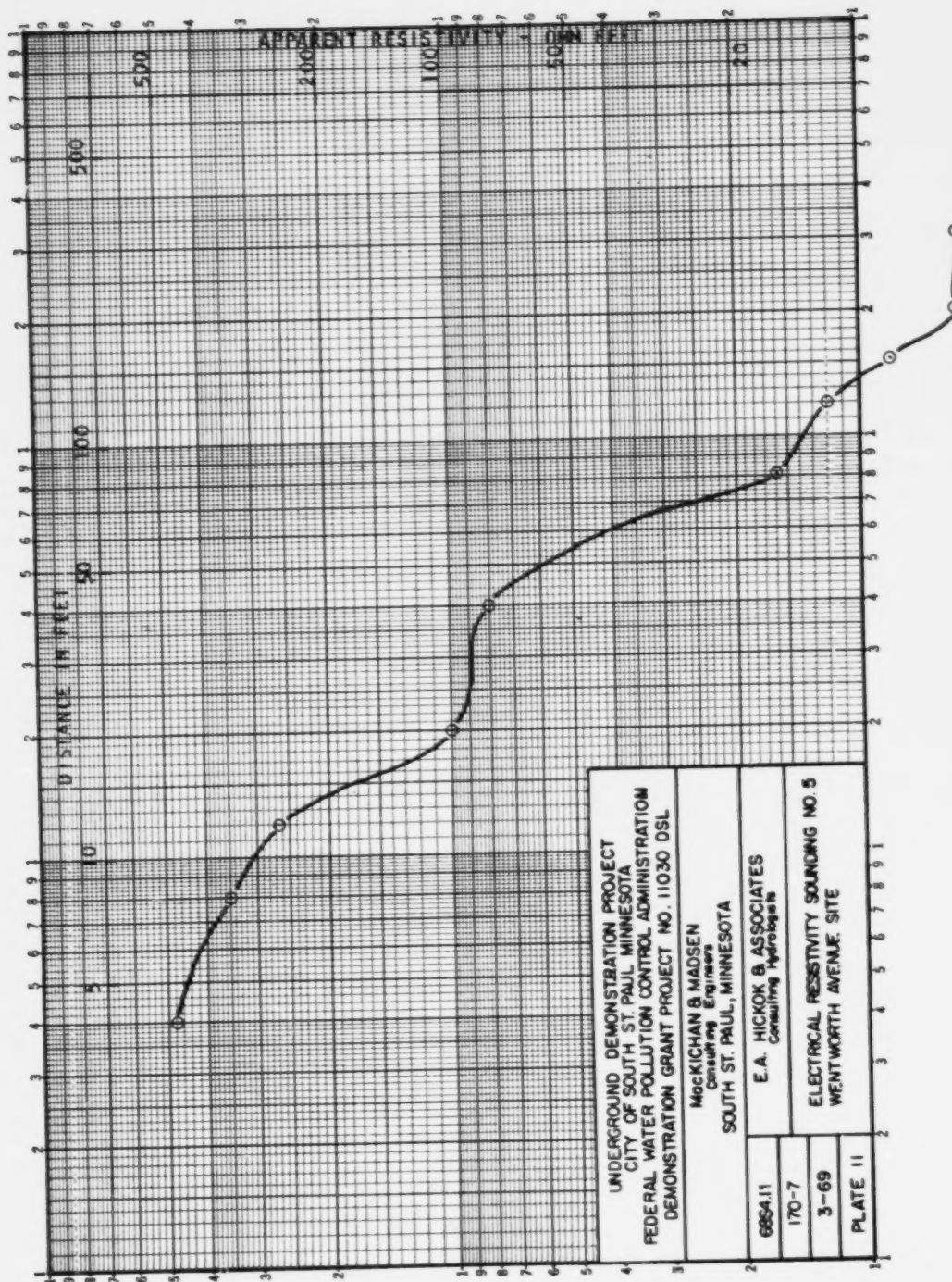


Figure 11.

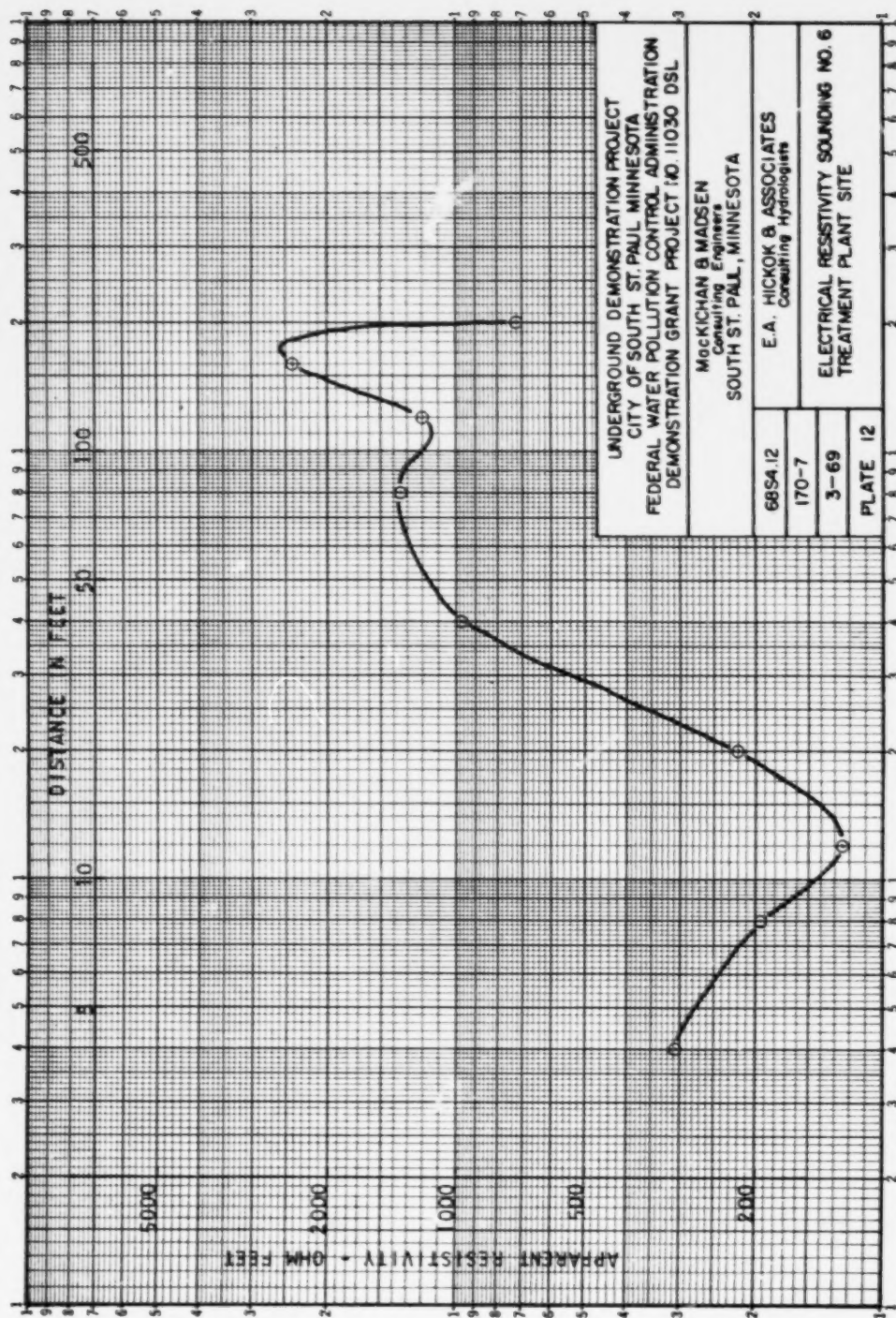


Figure 12.

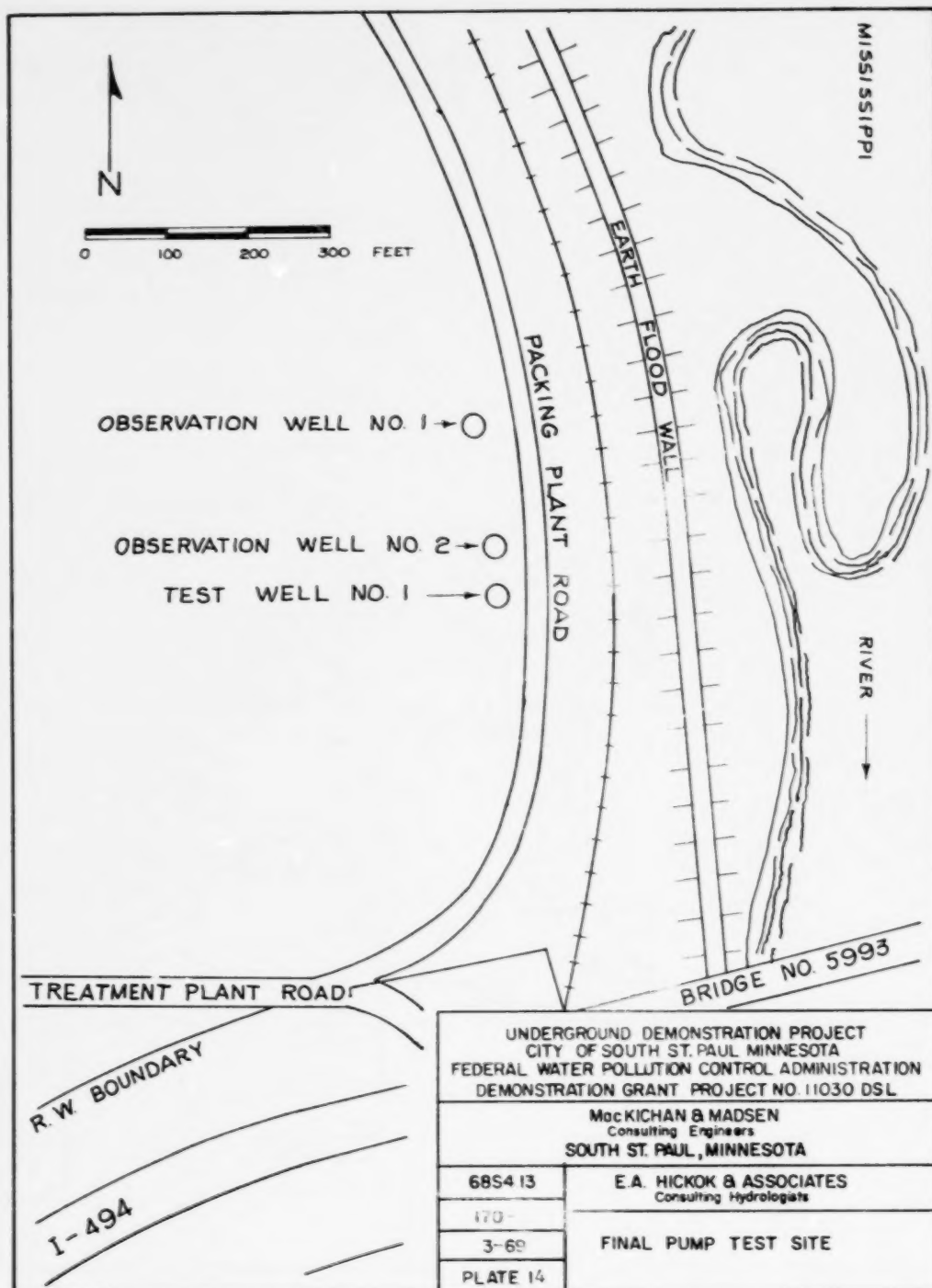


Figure 14.

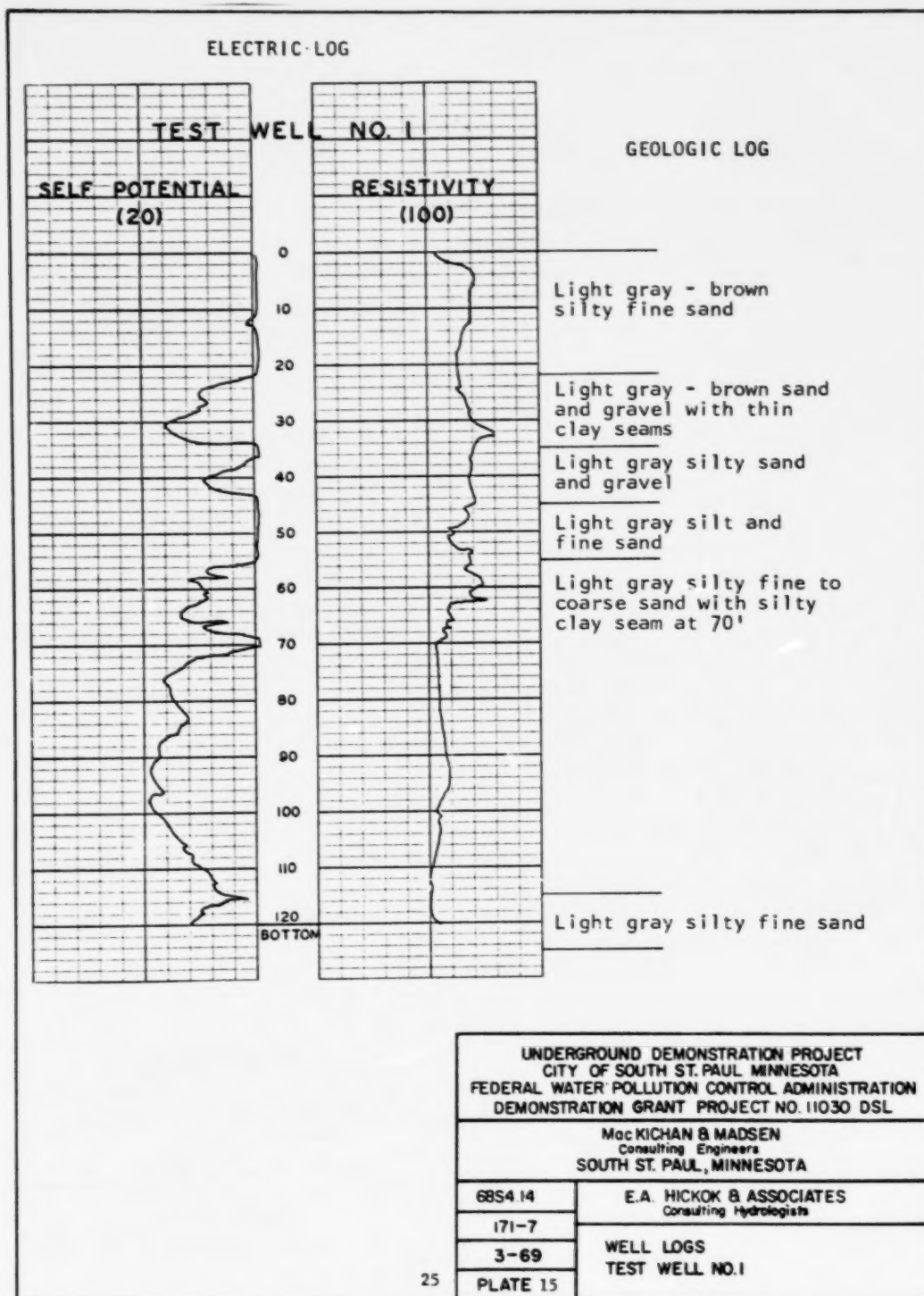
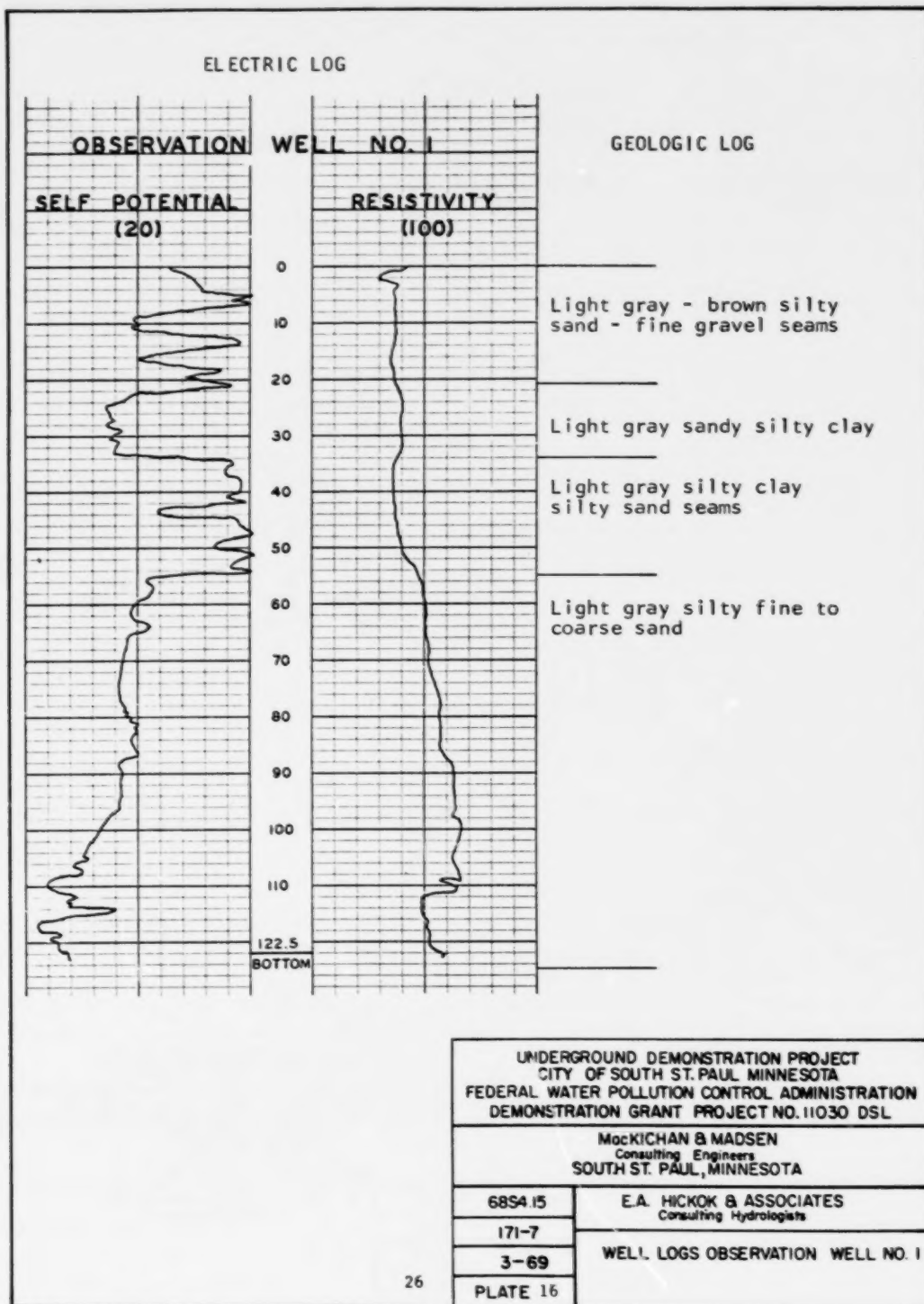


Figure 15.



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Figure 16.

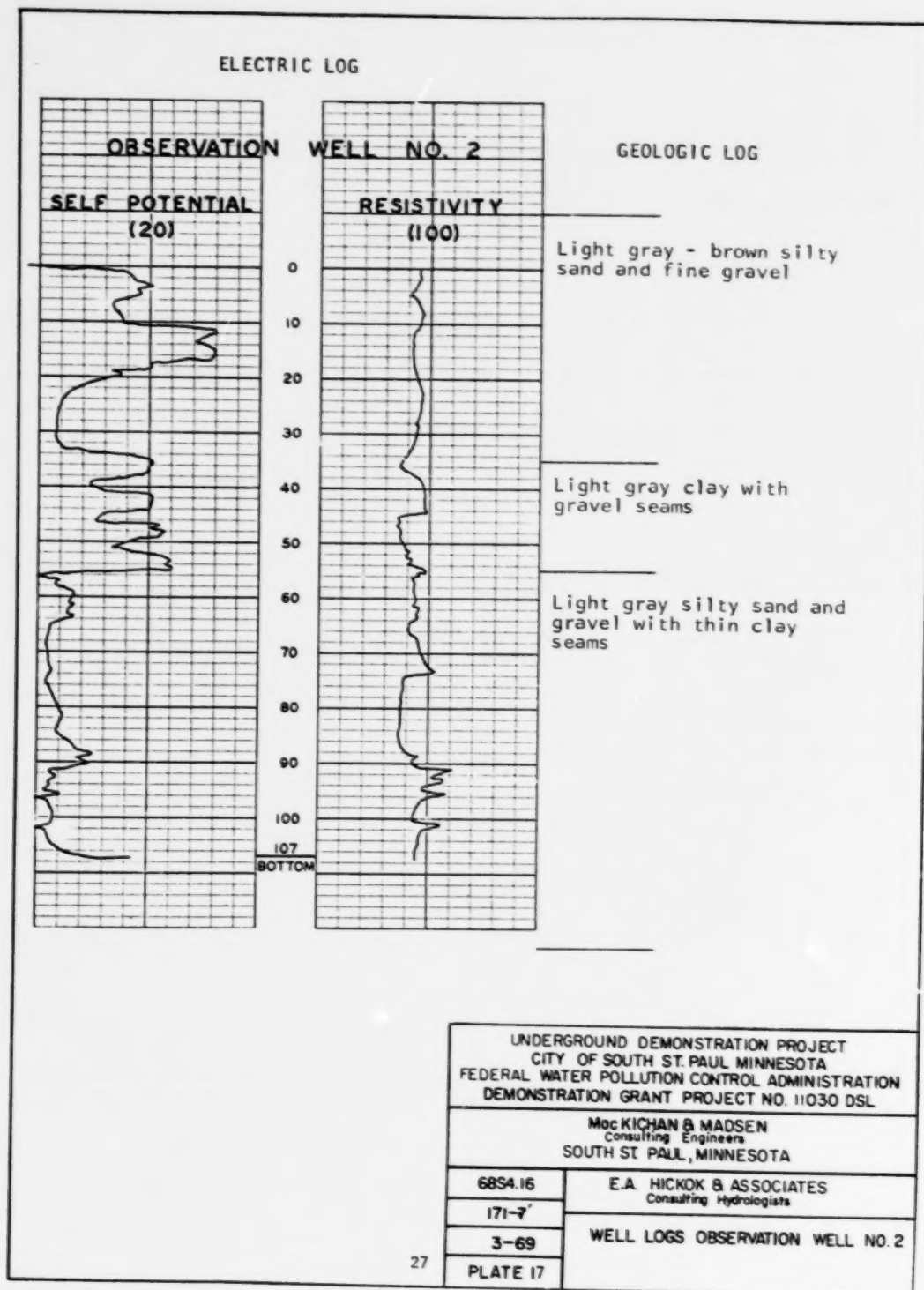


Figure 17.

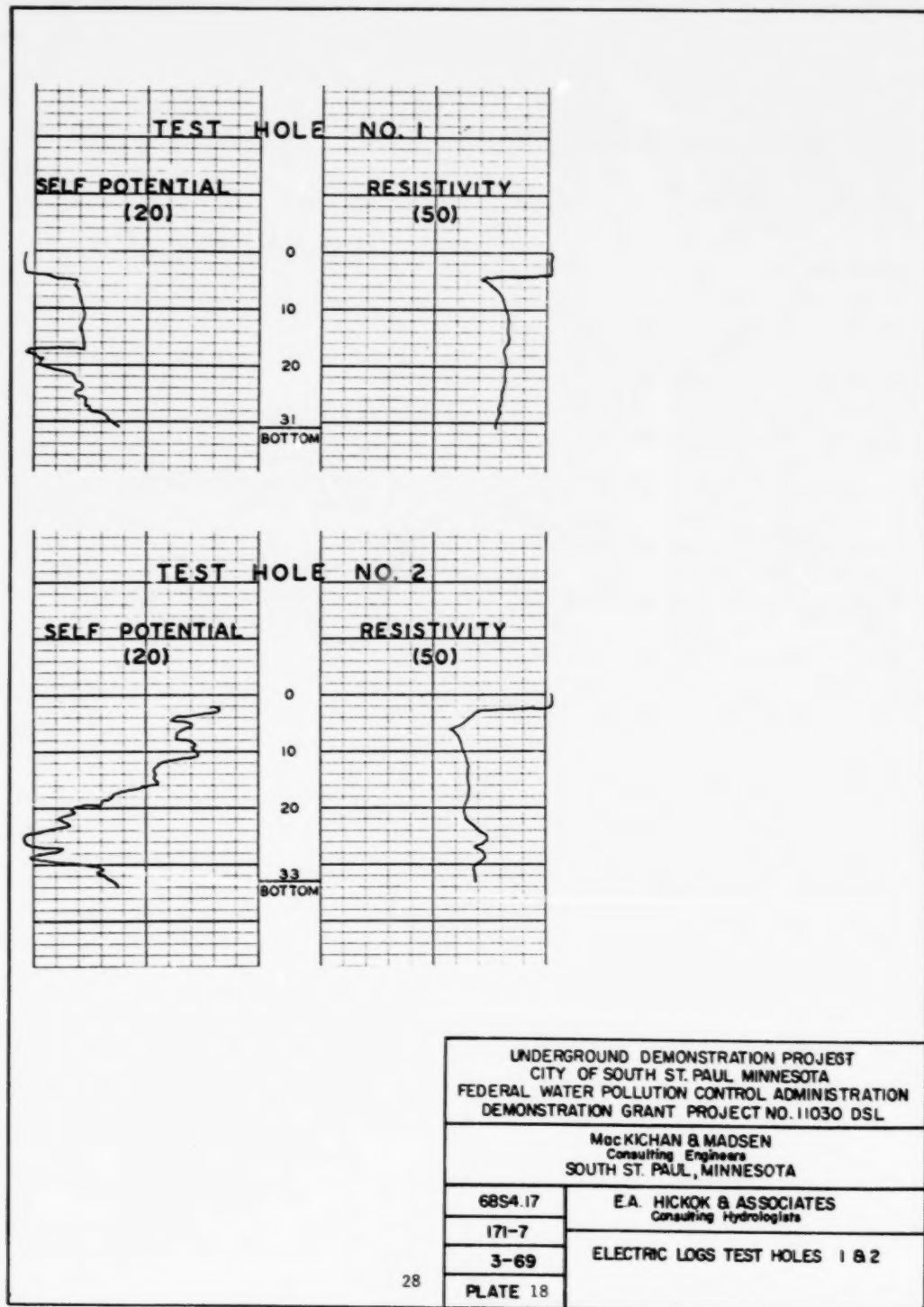


Figure 18.

The hole 50 feet north of TW-1 was designated Observation Well No. 1, OW-1 and the hole 150 feet north of TW-1 was numbered Observation Well No. 2, OW-2. Both observation wells were 4 inches in diameter. This layout is shown on Plate 14.

All three wells were electric-logged with a Widco Portalogger, as were TH-1 and TH-2. These logs, shown in Plates 15, 16, 17, and 18, record self-potential and resistivity, which with data from the driller's log, gives an accurate picture of subsurface materials and depths.

This subsurface information was used to position 10 foot long well-screens in each well in order to take advantage of the most permeable interval encountered in each hole. It is important in this procedure that each wellscreen intercept essentially the same aquifer. The electric logs indicated a good correlation between the intervals of maximum permeability, namely the best aquifer in the three wells.

After placement of the well screen and casing to each well, the hole was flushed of all drilling mud and developed for about four hours.

When all three wells, TW-1, OW-1, and OW-2, had been developed, preparations were made to pump TW-1 at close to maximum yield while observing water level fluctuations in all three wells. To assess the feasibility of underground storage of fluids, it is necessary to determine the rate at which a given aquifer will accept a charge and yield a discharge. Both events are functions of the same hydraulic properties that define the basic capacity of the aquifer. These are the coefficients of transmissibility and of storage. The first is defined as the quantity of water in gallons per day, GPD, that is transmitted, under a hydraulic gradient of one, through a vertical strip of aquifer one foot wide and extending the full thickness of the aquifer. The coefficient of storage is defined as the volume of water that an aquifer releases, or takes up, per unit surface area of aquifer per unit change in the component of head normal to the surface.

Field pumping tests, to determine the rate of decline or ground water levels during pumping, provide the most reliable means of determining these coefficients.

A test was conducted wherein TW-1 was pumped at a rate of 150 GPM for 24 hours while measuring water levels in TW-1 and both observation wells. The water analysis of Test Hole No. 1 pump test is shown in Table I. The pump test measurements are listed in Tables II, III, and IV, and plotted on graphs as Plates 19 and 20.

TABLE 1
WATER ANALYSIS
TEST HOLE NO. 1 PUMP TEST

Elapsed Time (Min.)	30	240	360	480	600	720	840	960	1080	1200	1320	1442
Sample No.	1	2	3	4	5	6	7	8	9	10	11	12
Analysis												
Alkalinity	395											380
Chlorides	10.0		10.0	10.0	12.5	10.0	10.0	12.5	10.0	10.0	10.0	10.0
Calcium Hardness	340											340
Total Hardness	230											230
Iron	4.30											4.49
Sulfates												
Manganese	0.60											0.63
Nitrite	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
Total Nitrate	0.03	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.01
Spec. Conductance	650	600	520	590	600	540	530	500	590	510	550	600
Copper												0.00
Coliform Bac.	Pos.											Neg.

Samples collected December 17, 1968
Analyzed December 19, 1968

TABLE II
PUMP TEST MEASUREMENTS

TEST WELL NO. 1 - PUMPING WELL

DECEMBER 6, 1968

STATIC WATER LEVEL - 15.93' BELOW GRADE - PUMP RATE - 150 GPM

TIME	ELAPSED TIME - MINUTES	WATER DEPTH - FEET	DRAWDOWN
10:35	0		
10:37	2	23.50	7.57
10:39	4	23.50	7.57
10:43	8	23.52	7.59
10:45	10	23.54	7.61
10:47	12	23.58	7.65
10:49	14	23.57	7.64
10:51	16	23.61	7.68
10:53	18	23.65	7.72
10:55	20	23.67	7.74
10:57	22	23.68	7.75
10:59	24	23.70	7.77
11:01	26	23.70	7.77
11:03	28	23.70	7.77
11:05	30	23.71	7.78
11:10	35	23.71	7.78
11:15	40	23.71	7.78
11:20	45	23.75	7.82
11:25	50	23.75	7.82
11:30	55	23.78	7.85
11:35	60	23.79	7.86
11:50	75	23.81	7.88
12:05	90	23.80	7.87
12:20	105	23.81	7.88
12:35	120	23.81	7.88
1:05	150	23.67	-
1:35	180	23.75	-
2:05	210	23.80	7.87
2:35	240	23.82	7.89
3:05	270	23.82	7.89
3:35	300	23.82	7.89
4:05	330	23.80	7.87
4:35	360	23.80	7.87
5:05	390	23.82	7.89
5:35	420	23.82	7.89
6:05	450	23.91	7.98
6:35	480	23.91	7.98
7:05	510	23.91	7.98
7:35	540	23.82	7.89
8:05	570	23.80	7.87

TIME	ELAPSED TIME - MINUTES	WATER DEPTH - FEET	DRAWDOWN
8:35	600	23.82	7.89
9:05	630	23.85	7.92
9:35	660	23.80	7.87
10:05	690	23.80	7.87
10:35	720	23.78	7.85
11:05	750	23.78	7.85
11:35	780	23.82	7.89
12:05	810	23.82	7.89
12:35	840	23.82	7.89
1:05	870	23.82	7.89
1:35	900	23.82	7.89
2:05	930	23.82	7.89
2:35	960	23.82	7.89
3:05	990	23.82	7.89
3:35	1020	23.82	7.89
4:05	1050	23.82	7.89
4:35	1080	23.82	7.89
5:05	1110	23.82	7.89
5:35	1140	23.82	7.89
6:05	1170	23.82	7.89
6:35	1200	23.91	7.98
7:05	1230	23.91	7.98
7:35	1260	23.91	7.98
8:05	1290	23.91	7.98
8:35	1320	23.91	7.98
9:05	1350	23.91	7.98
9:35	1380	23.91	7.98
10:05	1410	23.91	7.98
10:35	1440	23.91	7.98
10:47	1452	23.91	7.98

TABLE III
PUMP TEST MEASUREMENTS

OBSERVATION WELL NO. 1

50 FEET NNW OF TEST WELL NO. 1

DECEMBER 6, 1968

TIME	ELAPSED TIME - MINUTES	WATER DEPTH - FEET	DRAWDOWN
11:08	33	17.37	1.77
11:37	62	17.52	1.92
12:07	92	17.45	1.85
12:37	122	17.45	1.85
1:07	152	17.45	1.85
1:37	182	17.45	1.85
2:07	212	17.48	1.88
2:37	242	17.46	1.86
3:07	272	17.50	1.90
3:37	302	17.47	1.87
4:07	332	17.44	1.84
4:37	362	17.44	1.84
5:07	392	17.42	1.82
5:37	422	17.45	1.85
6:07	452	17.49	1.89
6:37	482	17.50	1.90
7:07	512	17.51	1.91
7:37	542	17.52	1.92
8:07	572	17.53	1.93
8:37	602	17.52	1.92
9:07	632	17.51	1.91
9:37	662	17.51	1.91
10:07	692	17.51	1.91
10:37	722	17.52	1.92
11:07	752	17.52	1.92
11:37	782	17.51	1.91
12:07	812	17.51	1.91
12:37	842	17.50	1.90
1:07	872	17.50	1.90
1:37	902	17.50	1.90
2:07	932	17.50	1.90
2:37	962	17.50	1.90
3:07	992	17.50	1.90
3:37	1022	17.50	1.90
4:07	1052	17.50	1.90
4:37	1082	17.50	1.90
5:07	1112	17.50	1.90
5:37	1142	17.50	1.90
6:07	1172	17.51	1.91
6:37	1202	17.51	1.91
7:07	1232	17.51	1.91

Pump Test Measurements - Observation Well No. 1 - Page 2

TIME	ELAPSED TIME - MINUTES	WATER DEPTH - FEET	DRAWDOWN
7:37	1262	17.51	1.91
8:07	1292	17.52	1.92
8:37	1322	17.53	1.93
9:07	1352	17.58	1.98
9:37	1382	17.56	1.96
10:07	1412	17.56	1.96
10:37	1442	17.56	1.96

TABLE IV
PUMP TEST MEASUREMENTS

OBSERVATION WELL NO. 2

150 FEET NNW OF TEST WELL NO. 1

DECEMBER 6, 1968

TIME	ELAPSED TIME - MINUTES	WATER DEPTH - FEET	DRAWDOWN
11:09	34	17.18	1.28
11:43	68	17.22	1.32
12:10 PM	95	17.22	1.32
12:40	125	17.25	1.35
1:10	155	17.25	1.35
1:40	185	17.05	1.15
2:10	215	17.30	1.40
2:40	245	17.28	1.38
3:10	275	17.27	1.37
3:40	305	17.28	1.38
4:10	335	17.30	1.40
4:40	365	17.28	1.38
5:10	395	17.27	1.37
5:40	425	17.28	1.38
6:10	455	17.29	1.39
6:40	485	17.28	1.38
7:10	515	17.28	1.38
7:40	545	17.31	1.41
8:10	575	17.34	1.44
8:40	605	17.34	1.44
9:10	635	17.34	1.44
9:40	665	17.33	1.43
10:10	695	17.31	1.42
10:40	725	17.33	1.43
11:10	755	17.33	1.43
11:40	785	17.33	1.43
12:10	815	17.33	1.43
12:40	845	17.33	1.43
1:10	875	17.33	1.43
1:40	905	17.33	1.43
2:10	935	17.33	1.43
2:40	965	17.33	1.43
3:10	995	17.33	1.43
3:40	1025	17.33	1.43
4:10	1055	17.33	1.43
4:40	1085	17.33	1.43
5:10	1115	17.33	1.43
5:40	1145	17.33	1.43
6:10	1175	17.33	1.43
6:40	1205	17.33	1.43
7:10	1235	17.33	1.43

Pump Test Measurements - Observation Well No. 2 - Page 2

TIME	ELAPSED TIME - MINUTES	WATER DEPTH - FEET	DRAWDOWN
7:40	1265	17.33	1.43
8:10	1295	17.33	1.43
8:40	1325	17.33	1.43
9:10	1355	17.33	1.43
9:40	1385	17.33	1.43
10:10	1415	17.33	1.43
10:40	1445	17.33	1.43

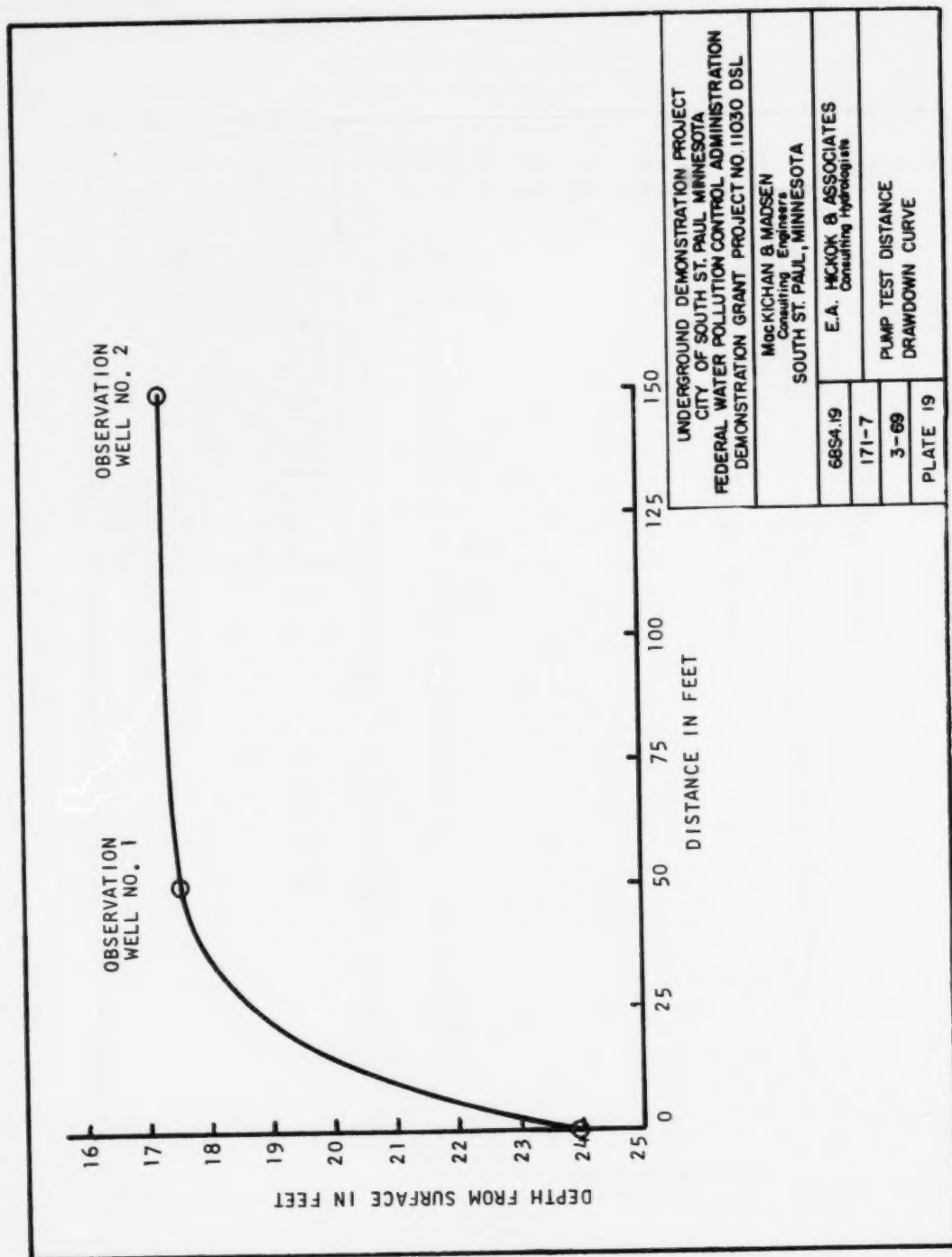


Figure 19.

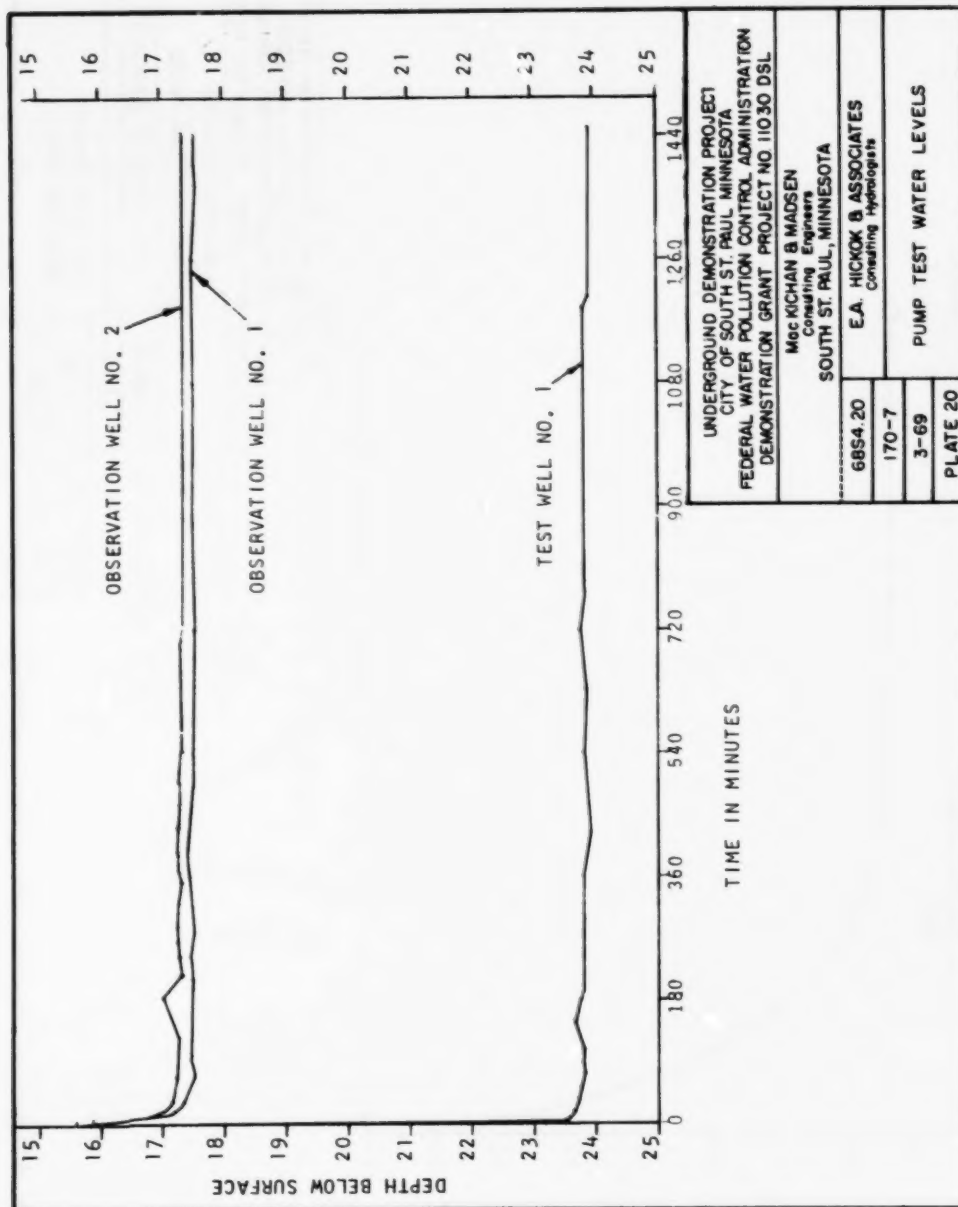


Figure 20.

Computations of the test data indicate a coefficient of transmissibility of 38,500 GPD per foot, a coefficient of storage of 2×10^{-4} , and a specific capacity of 18.8 GPM per foot of drawdown for that particular well size and screen length.

Conclusions are based on the assumption that the subsurface conditions found at drill hole TW-1 are indicative of the actual existing condition for a radius of about 500 feet around TW-1.

SECTION V

SUITABILITY OF TEST WELL SITE

The desirable characteristics of a site for demonstration for the underground detention method includes ease of access to existing sewers; a sufficient thickness, permeability, and capacity in subsurface formation to permit storage and demonstration of the method without contamination of existing aquifers; a surface area of sufficient size for the necessary settling basin and facilities for separation of solids prior to injection of storm or combined solids underground; and a sufficiently low elevation to make use of gravity flow.

Results of field investigations showed TW-1, the site of which the pump test was conducted, appeared to fulfill all these requirements and it is fairly remote from human activity so as to cause a minimum of disturbance if the demonstration project would have been continued into Phase II. This site is also immediately upstream from the Sewage Disposal Plant.

The results, based on the actual pump test at TW-1 and the observations made at OW-1 and OW-2 (see Table V), show that for demonstration purposes a series of five wells spaced at 200 foot intervals with pumping capacities of 200 GPM each could be constructed. See Plate 21.

Plate 22 is a detail drawing of a theoretical well plan that could be used for each of the 5 well sites.

TABLE V
PROJECTED OPERATING CONDITIONS
FIVE WELLS AT 200 FOOT SPACINGS
RECHARGE RATE 200 GPM PER WELL
SPECIFIC CAPACITY 25 GPM PER FOOT

Well Numbers	1	2	3	4	5
1	--	1.6	1.5	1.4	1.3
2	1.6	--	1.6	1.5	1.4
3	1.5	1.6	--	1.6	1.5
4	1.4	1.5	1.6	--	1.6
5	1.3	1.4	1.5	1.6	--
	—	—	—	—	—
Interference Cone	5.8	6.1	6.2	6.1	5.8
Recharge Cone	8.00	8.0	8.0	8.0	8.0
	—	—	—	—	—
Total Cone Height	13.8	14.1	14.2	14.1	13.8
S.W.L.	15.6	15.6	15.6	15.6	15.6
	—	—	—	—	—
Water Depth	1.8	1.5	1.4	1.5	1.8

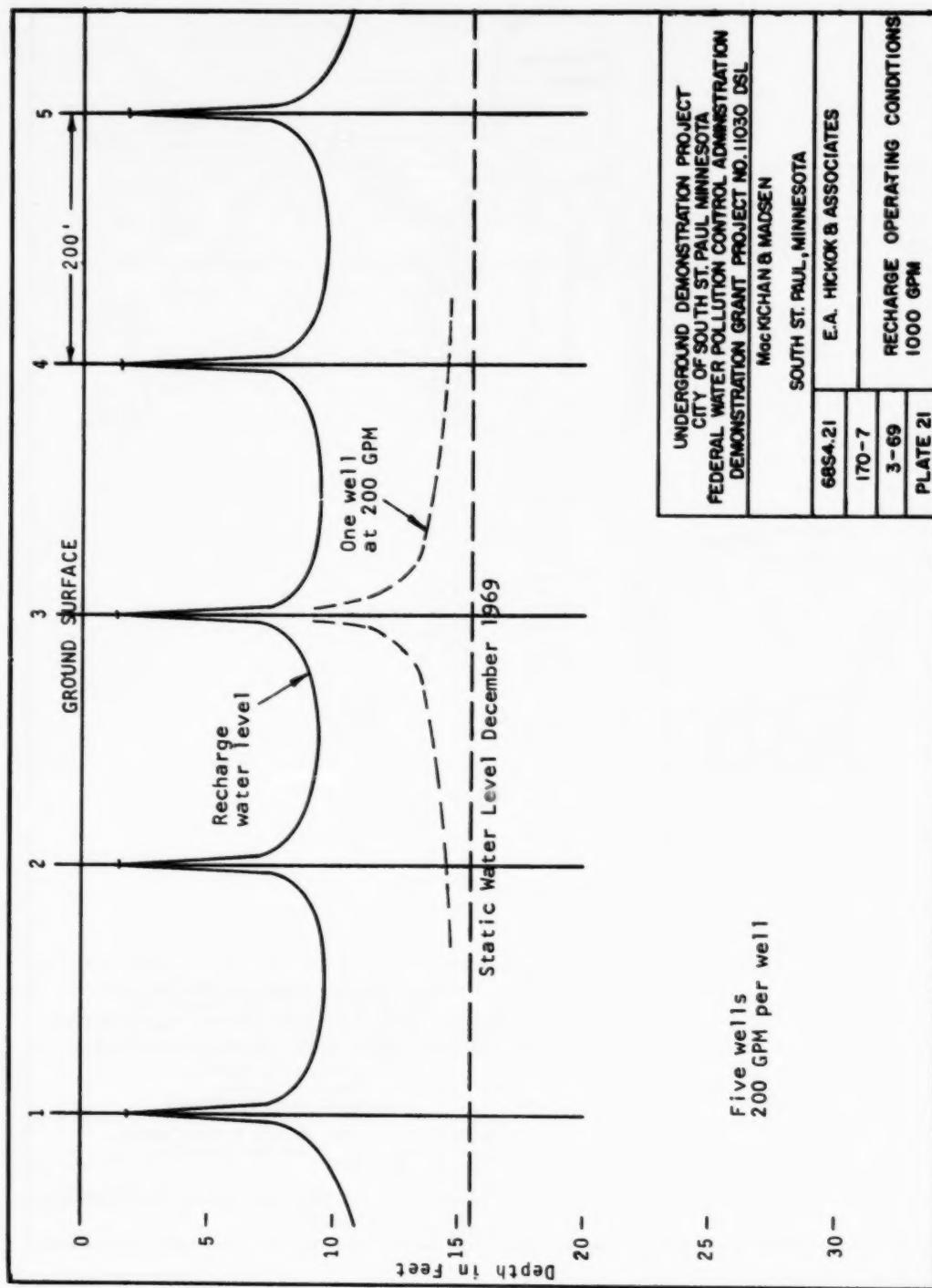


Figure 21.

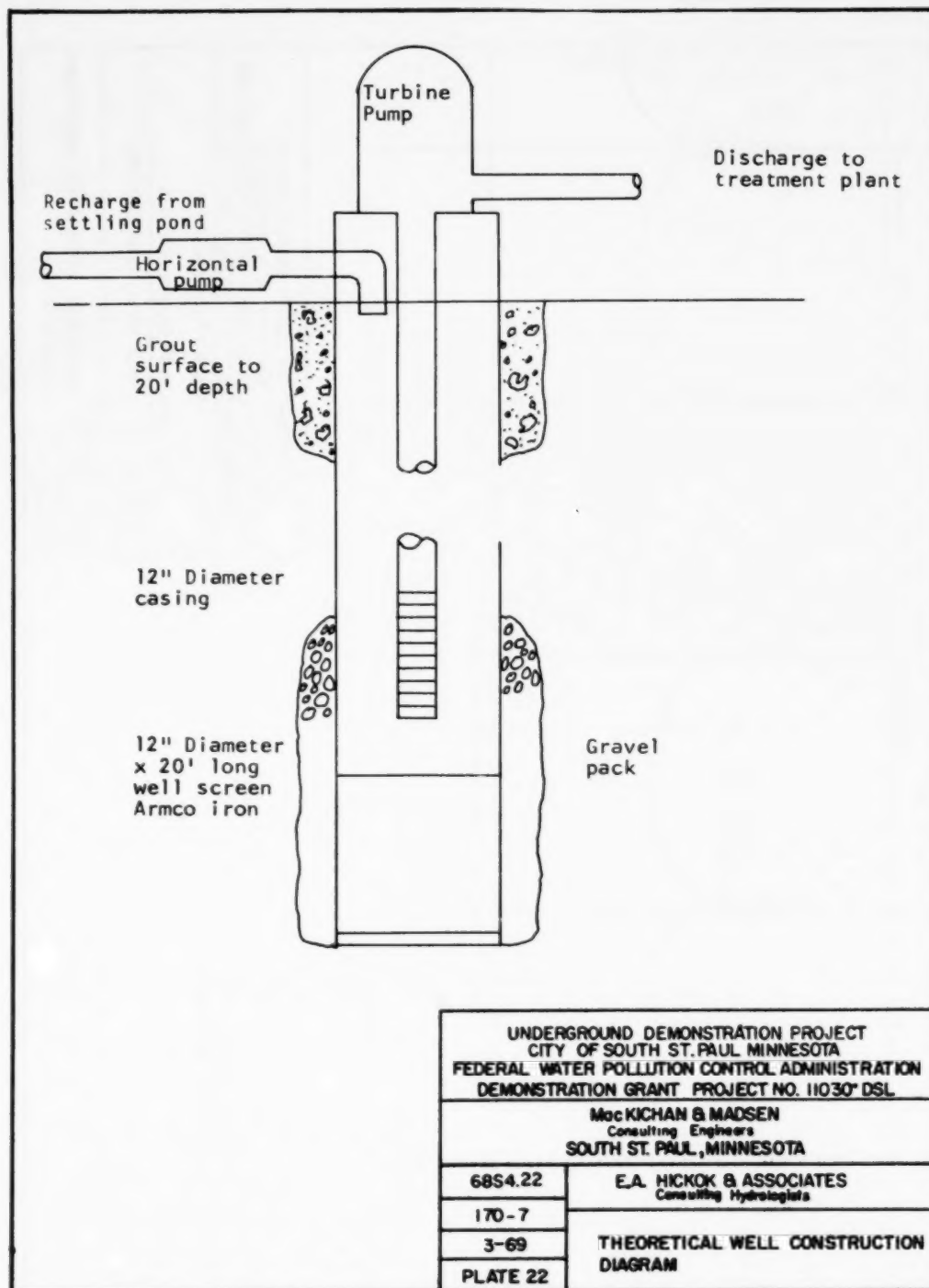


Figure 22.

SECTION VI

SOLIDS SEPARATION INVESTIGATION

Included in the scope of work for Phase I of the Demonstration Project was an investigation, analysis and discussion of methods of solids separation which might be used for storm and combined sewage in Phases II and III, prior to injection of the effluent underground.

There are several design parameters which have to be established before final design or recommendations for separation for solids can be made. Included in these parameters are such things as the rate in total volume of storm flow to be handled, the rate at which the storm flow can be injected into the underground strata and the solids volume contained in the storm or combined flow in terms of gradation of solids and type of solids. Also, a specific limitation on solid size of concentration would have to be determined for the hydraulic volumes to be injected in the subsurface strata.

For injection of the storm or combined flow into the subsurface strata there are two extreme conditions, the first being that all storm-combined flow be stored in surface reservoirs and withdrawn at a constant rate for injection into the subsurface strata, and the second that all storm and/or combined flow would be injected directly into the subsurface strata without storage. There would, of course, be an infinite number of variations between the two extremes. A method which could be practiced for solid removal is sedimentation or settling of solids in detention basins with or without the flow receiving chemicals to assist in flocculation and sedimentation. Sedimentation is accomplished with a due consideration to velocity control, surface settling rate and detention time. Additional solids removal could be separated by straining or filtration of water which could include use of sand filters or fine screens with a certain size limitation of solids passage for the screen. A combination of sedimentation and screening could be practiced depending upon solids concentration or particle size limitation for wastewater to be injected into the subsurface stratus.

The preliminary investigation included the assumption that all or portions of the storm and/or wastewater would be stored in surface reservoirs with solids separated prior to injection into the underground strata. The problem then resolves to methods of removing solids from surface reservoirs which settle during storage plus possible filtering or settling of effluent from the basins to a specific quality prior to discharge to the injection wells plus handling of solids separated in the process.

Soils can be classified by particle size as follows:

Gravel	-	larger than 2.0 millimeters
Coarse sand	-	2.0 to 0.42 millimeters
Fine sand	-	0.42 to 0.05 millimeters
Silt size	-	0.05 to 0.005 millimeters
Clay size	-	smaller than 0.005 millimeters

A 200 mesh U. S. Standard sieve has an opening of 0.074 millimeters. Classification of fine sand can be made to 0.074 millimeters rather than 0.05 millimeters, as previously noted. The law of averages narrows the usual range of variation of the true specific gravity of soils to values between about 2.55 and 2.75.

Specific gravity of organic solids in combined sewage flows can range from less than 1.0 (floatables such as grease) to 2.65 (soil).

Sedimentation efficiency depends upon many variables such as temperature of water, specific gravity of materials in suspension and size and shape of suspended particles.

Sedimentation of 4 micron clay Sp. gr. 2.65 in 10° C (50° F.) water can be accomplished at an overflow rate of 52 gallons/S.F./day. At water temperature of 32°F. the overflow rate would be 38 gallons/S.F./day.

Sedimentation alone for organic solids removal cannot be accomplished to consistently satisfy a maximum particle size of 5 microns. Mechanical screening with micro-screens can, however, be done, as commercially available screens have a screening capability of 23 to 25 microns. It is therefore concluded and recommended that if waters to be injected into the underground strata contain organic solids, provisions be made in the solids removal system to include or add chemical mixing and flocculation facilities followed by sedimentation basins and water filtration facilities, such as rapid sand, mixed or dual media filters. It is noted that should chemical treatment be practiced, the chemical balance of the water may have to be re-established prior to subsurface injection.

Other recommended facilities should include trash screens and facilities for grit removal.

Trash screens recommended are mechanically cleaned type with screen with 1-inch clear openings. Screen recommended is a front cleaned cable operated unit. Debris removed should be hauled to a sanitary landfill. All flow should be screened when received.

Grit removal units recommended would be sized to remove 100% of 150 mesh (104 micron) material. Facilities proposed would be two units, the first sized to remove 100% of 35 mesh grit and sized at 80,000

gallons/S.F./day. Grit would be removed from the basin with either a dredging pump or overhead hoist with a clam shell. The second or subsequent unit would be sized for 100% removal of 150 mesh materials. Basins would be sized at 14,000 gallons/S.F./day and be equipped with traveling bridge collectors with bottom scrapers and water jets to remove settled material. Settled material to be pumped to a cyclone separator with effluent to sanitary sewer. Grit recovered in either or both units to be used as fill material, or land filled as organic contents dictates.

Storage basins would be sized to contain all waters until disposal. Size of basins would be dependent upon inflow and outflow rates. Storage basins would be drained to a sump for return as settled materials to the grit recovery units. Provisions should be made either to allow flushing of settled material to the sump or to allow operation of dragline or front-end loader to remove settled materials. Dependent upon settleability of solids, provisions could be made to withdraw pond effluent for subsurface injection or subsequent chemical flocculation, sedimentation, filtration, and injection. Should pond effluent be withdrawn for direct injection, scum baffles and/or fine screens should be provided to prevent floating material from entering the injection system.

Chemical mixing and flocculation should be accomplished in a quick mix basin with mechanical mixer, 30 seconds minimum detention, to rapidly dispense chemicals throughout the water. Mixing should be followed by flocculation basins sized to provide 30 minutes detention with velocity control by mechanical agitators moving 0.5 to 2.0 feet per second. Lime and alum chemical feed equipment and chemical storage facilities are required plus chlorination equipment for possible use in applying chlorine to raw water or prior to filtration.

Sedimentation after chemical flocculation should be sized on 4 hours detention time. Basins should be equipped with mechanical sludge removal equipment. Quite possibly, basins used as grit recovery units could be used as sedimentation basins during periods of no raw inflow. Sludge from basins could be discharged to the sanitary system for removal, at the sanitary treatment plant, and possibly be beneficial to sanitary sedimentation due to coagulants contained.

Filtration units should follow sedimentation tanks to insure removal of all 5 micron and larger particles. Mixed media or dual media filtration units operated at 5 gpm/S.F. should provide adequate solids removal. Backwash water from units should be returned to storage ponds.

A 1,000 gpm pilot plant for solids recovery and injection was considered. The following solids recovery facilities are recommended for consideration.

Trash screen, $\frac{1}{2}$ inch clear openings, hand cleaned, size for a maximum flow through velocity of 3 fps.

Grit Recovery, size for 100% removal of 150 mesh (104 micron) surface settling rate 14,000 gallons/S.F./day. Use tank 10' x 10.3' x 7.0 SWD with hopper bottom with solids removed from the bottom either by gravity to a grit washer or pumped to a grit washer or cyclone. Dispose of solids in landfill.

Storage Basin. At a flow rate of 1,000 gpm, 1,440,999 gallons/day would be handled. For removal of all inorganic materials 5 microns and larger with specific gravity of 2.65 at a water temperature of 32°F, a settling basin of 38,000 square feet is required. Basin should be baffled to prevent short circuiting of flow with a maximum basin water velocity of 0.5 feet per minute. Basin effluent should pass over a weir at a maximum flow rate of 10,000 gallons/L.F./day. A scum baffle should be provided at the weir to prevent floatable materials leaving the basin.

Should chemical flocculation, sedimentation and filtration be required to remove all particles larger than 5 microns, the following is suggested.

Quick Mix - Use 500 gallon tank with mixer. Add synthetic polymer as flocculant.

Flocculation Unit - Non-mechanical, use baffled channel to provide 1 fps velocity for 30 minutes. Use 1.5' water depth x 3.0' wide channel with alternate 1.5' x 1.5' baffle plates @ 1.5' $\emptyset$ center to center spacing.

Sedimentation Unit - Non-mechanical, use basin 10' deep x 20' wide x 160' long with scum baffle and 72 L.F. of overflow weir. Drain to sump for solids return to grit unit or to sanitary sewer.

Filtration Unit - Use dual media or mixed media loaded at 5 gpm/S.F. Return backwash water to storage pond for subsequent settling. Consider automatic back gravity filter such as Eimco Type SUBG.

Storm water solids captured by screens or settling can possibly be used as fill material. Solids recovered from combined sewer would have to be land filled or disposed of with other city sanitary waste solids.

Materials flushed from filters could be re-cycled through the grit-silt removal or be discharged to the sanitary system.

SECTION VII

DISCUSSION OF RESULTS

Phase I of this report has only attempted to locate a site suitable for subsequent testing. In order to ascertain the feasibility of the method under operating conditions and to determine cost factors, Phases II and III would have to be completed. Due to the inadequate availability of underground storage capacity, it was decided not to proceed with Phases II and III.

A recently completed study on combined sewage system separation and treatment for South St. Paul, prepared by Henningson, Durham & Richardson, evaluated 4 alternate methods of handling the pollution problems caused by discharging combined storm water and wastewater into the Mississippi River during periods of high surface runoff.

The methods considered in the study are referred to as Detention & Treatment; Schemes A, B, and C, and Total Separation.

The study recommends Total Separation of the systems based on economics and the lack of available land for surface detention basins. See Appendix A for data relating to the "Combined Sewerage System Separation and Treatment South St. Paul, Minnesota" study.

SECTION VIII

ACKNOWLEDGEMENT

The City of South St. Paul, Minnesota, would like to acknowledge the people and organizations whose efforts and knowledge are incorporated into this study.

City of South St. Paul

Mr. John P. Badalich
Mr. Donald R. Peterson
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Mr. Richard A. Hudak, Engineering Administrator

Consultants

MacKichan & Madsen, Consulting Engineers
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U.S. Environmental Protection Agency
Minnesota-Wisconsin District Office
Mr. Clarence C. Oster

SECTION IX

GLOSSARY OF PERTINENT TERMS

Geophysical - The study of the physical characteristics and properties of the earth. In this study dealing primarily with the use of seismic and electrical resistivity methods.

Geological - The science which deals with the history of the earth and the materials which it is composed of. In this study dealing primarily with the alluvial strata.

Hydrological - The branch of hydrology that treats ground water; its occurrence and movements; its replenishment and depletion; the properties of rocks which control ground water movement and storage; and the methods of investigation and utilization of ground water.

Combined Sewage System - A system whereas during dry weather conditions, the sewer conduits direct all sanitary wastewater to the sewage treatment plant during surface runoff conditions due to precipitation or melting snow, the water is collected in the same sewer conduit as the sanitary wastewater. All flows exceeding capacity of the conduit leading to the treatment plant overflow into a pond, lake, stream, river, etc.

Piezometer Wells - An instrument for measuring pressure head in a conduit, tank, soil formation, etc. In the case of the study, the piezometer wells were used to observe the fluctuation of the water level in the soil formations to be tested, and also to allow sampling of the water for analysis purposes.

Chemical Flocculation - The introduction into a sewer treatment process a coagulant to form small gelatinous masses to aid in solids removal.

Electrical Resistivity Soundings - A method by which earth and rock materials can be identified by their reaction to the flow of a direct current of electricity. This is an action of electrolytic nature in which moisture in the soils and rocks, together with dissolved impurities, gives to the several materials characteristic resistances to the current flow. These characteristic resistances may be used for locating and identifying subsurface formations. Ordinary moist soils containing moderate amounts of clay or silt, with some electrolytic agent more or less active, have a comparatively low resistance. In contrast, sand, gravel, extremely dry, loose soils and solid rock usually have high resistivity values.

SECTION X

APPENDICES

- A. Combined Sewage System Separation and Treatment, South
St. Paul, Minnesota.

APPENDIX A

Separation and Treatment of Combined Sewerage System, South St. Paul, Minnesota, by Henningson, Durham and Richardson, Omaha, Nebraska.

TOPICAL SUMMARY

1. Existing sewage collection system is grossly overloaded for storm water flow in many locations.
2. The dilutional effect of storm water on the sanitary sewage during times of overflow to river is insufficient to abate pollution, particularly from the standpoint of disease bacteria.
3. Either Total Separation - with storm water discharging to the river and all sanitary flow treated at central treatment plant before discharge to the river - or sufficient treatment of the combined sewage is required to satisfactorily abate pollution of the river.
4. Total separation and three methods of treatment were studied in detail. Other possible methods of treatment were discarded as not economically feasible.
5. Cost comparisons are as follows:

Detention and Treatment	-	Method "A"	\$4,838,367
Detention and Treatment	-	Method "B"	\$5,609,390
Detention and Treatment	-	Method "C"	\$3,558,782
Total Separation	-		\$4,020,745
6. The additional storm sewer pipe required to bring the storm drainage system up to a theoretical ten-year storm capacity are estimated at a cost of \$960,372. This additional sewer pipe is not required for separation and is illustrated for informational purposes only.
7. Total separation is recommended for the following reasons:
 - (a) Treatment under any of the three (3) methods would create odor problems due to solids residue in detention ponds along the river. These detention ponds would create potential blight areas, particularly to new industrial expansion along the river front.
 - (b) To create detention basins of the size required away from the populated area is totally unfeasible due to high costs of extremely large trunk outfall sewers required.

- (c) Treatment under the least costly Method "C" would only be partial. At times of low storm flow, the dilution factor would not be high enough to adequately reduce BOD and solids content.

Note: Construction cost estimates have not been escalated for future materials or labor costs.

- (d) Elimination of storm water surge to the existing sewage treatment facilities will reduce treatment costs and allow the present plant to operate more efficiently.
- (e) Separation of storm water from the sanitary sewer system will eliminate public health hazards of back-up and overflow of sanitary sewage into basements and onto streets.
- (f) Total separation costs are lower than two of the three methods of treatment and only 6% higher on a 20-year annualized cost basis than the third Method "C".
8. Sources of financing for the separation project are recommended as follows:

FWPCA - Sewer Separation Demonstration Grants
 DHUD - Community Facilities Construction Grants
 DHUD - Urban Renewal Construction Grants
 DHUD - Industrial Development Grants
 General Obligation Bonds
 Sewer Use Revenue Bonds
 State Aid - Street Improvements

9. The following is the recommended expenditure schedule for the construction of storm sewers and sanitary sewers, which will be required in order to provide separation of storm flows from sanitary flows.

Summary of Costs on a Yearly Basis:

<u>Year</u>	<u>Capital Improvements</u>	<u>Engin., Admin., Legal, Interest & Contingencies</u>	<u>Total Project Cost</u>
1970	\$206,180	\$ 51,545	\$ 257,725
1971	338,518	84,629	423,147
1972	650,577	162,644	813,221
1973	653,722	163,430	817,152
1974	682,071	170,518	852,589
1975	685,529	171,382	856,911
			<u>\$4,020,745</u>

10. The construction costs of sewers within State Aid Street Improvement Boundaries for 1970 equals \$107,985. The Urban Renewal Grant for sewer construction in 1971 approximately equals \$159,000.

ALTERNATE METHODS OF TREATMENT

TREATMENT OF COMBINED SEWAGE

1. General

Three different schemes of ponding have been considered. Combining these different schemes for various systems is not considered practical since each proposes allowing the passage of different basic mixtures of combined sewage to the river. In each scheme, the flows to the river are to be chlorinated. Two of the three schemes provide for treatment of temporarily stored combined sewage at the existing treatment plant. The third scheme provides for chlorination and a short detention prior to passage into the river.

2. Scheme A

In this scheme the combined sewage up to a maximum of two and one-half ($2\frac{1}{2}$) times the average daily flow will be routed directly to the treatment plant for treatment. Flows between two and one-half ($2\frac{1}{2}$) and ten (10) times the average daily flow will be routed to a pond for temporary storage. Flows greater than ten (10) times the average daily flow will be routed directly to the river after chlorination. Assuming that the average daily flow of domestic sewage contains 220 ppm BOD and the storm runoff contains 20 ppm BOD that combined sewage routed directly to the river would contain 35 ppm BOD. Currently the BOD for storm water is higher than 20 ppm. Any combined sewage flows routed to the river will be chlorinated. The symbol Qsa is used herein to indicate $2\frac{1}{2}$ times the average daily sewage flow (250%). 3 Qsa represents 750% of the average daily flow or the amount of storm flow required to dilute the sanitary flow. The total wet weather flow is identified as $4 \text{ Qsa} = \text{Qsa} + 3 \text{ Qsa} = 10$ times the average daily flow.

Controls will be liquid level sensing controlled gates. When the flow reaches a depth that produces a flow of $2\frac{1}{2}$ times average daily flow the gate to the pond opens, the gate to the interceptor closes, and the pumps are started. When the flow reaches a depth that produces a flow of 10 times average daily flow the gate to the river opens, the gate to the pond closes, and the chlorinator is started. The reverse control operation will take effect as the flow diminishes.

Ponds are sized to contain a constant rate equal to 10 times the average daily flow for a period of 48 hours during which time the rate of pumping equals the plant hydraulic capacity.

The combined sewage left in the pond after a rainstorm will be pumped to the plant for treatment during the evenings and week-ends. The combined rates of pumping from the ponds will be controlled so as not to exceed the hydraulic capacity of the plant.

Since there are no available areas to construct ponds to serve the Wentworth Avenue and Grand Avenue systems, they have not been included for ponding of combined sewage. Also, these two systems are located a considerable distance from the nearest downstream pond area and would require more than 7,000 feet of new combined sewer line to connect them to the nearest pond area.

Plate 23 shows a schematic layout of this proposed combined sewage pond system.

The following table indicates the basic design required for ponding of combined sewage under Scheme A for those drainage systems under consideration:

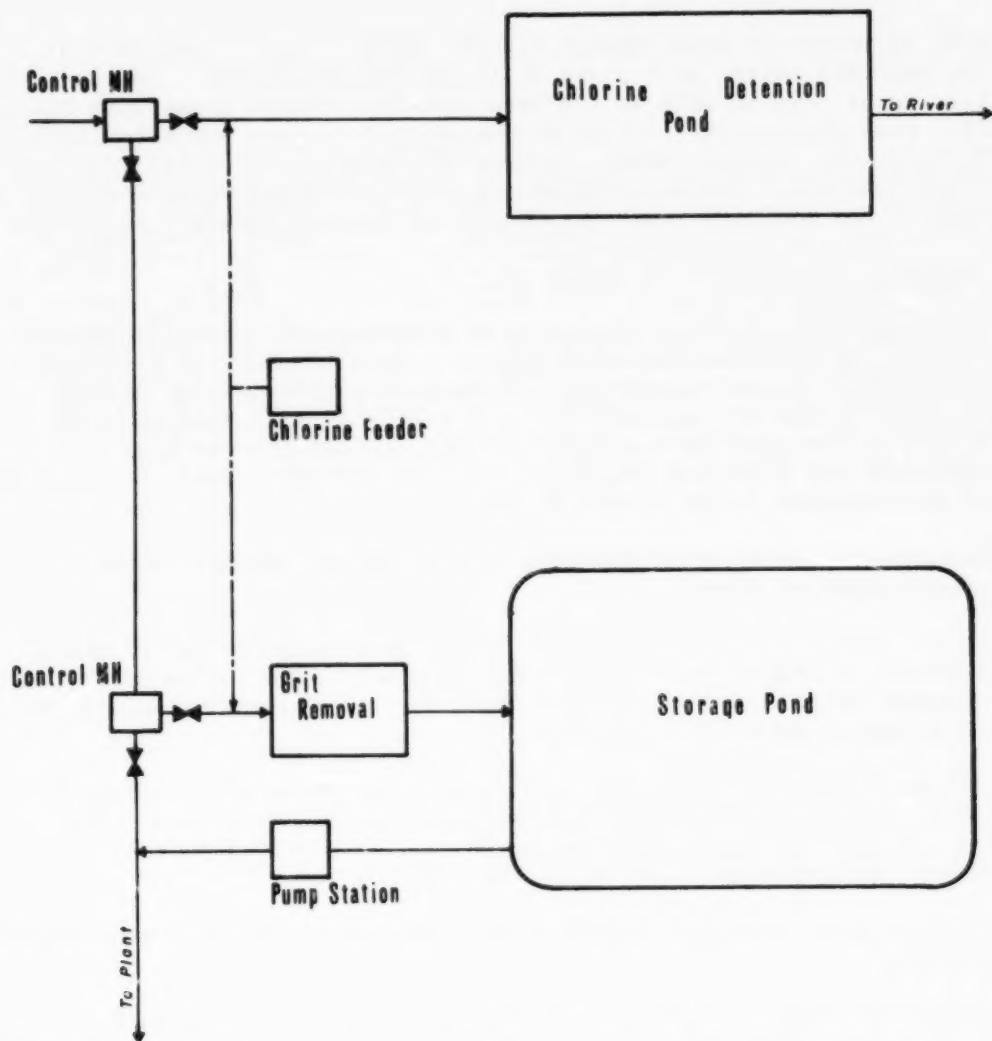
	<u>System</u>			
	<u>Stanley Ave.</u>	<u>Central Ave. & Simons Ravine</u>	<u>Sixth Street</u>	<u>Maltby Street</u>
2.5 x Avg. Daily Flow cfs	0.780	4.585	1.897	6.914
10.0 x Avg. Daily Flow cfs	3.120	18.340	7.560	27.660
Max. Flow with 10 Yr. Rain Intensity cfs	114.63	358.39	202.09	772.85
Pond Volume - Ac-Ft.	11.14	69.11	29.17	104.20
Pumping Capacity Max. GPM	490	3040	1285	4600
Chlorination Capacity Lbs/Day	800	2500	1400	5650
Grit Removal Chamber Sq. Ft.	144	345	350	1275
Time Required to empty pond Hrs.	145	145	145	145
Line Size to Pond In-Dia.	15	30	18	30
Line Size to Overflow In-Dia.	15	30	18	30
Chlorination Pond Size Length Ft.	513	713	748	865

It will be necessary to construct additional interceptor line capacity and increase existing lift station capacity in order to pass the flows during periods when the ponds are being emptied by pumping. The following additions to the present interceptor system will be required under this scheme.

a. Increase pumping capacity from 350 gpm to 800 gpm in the existing lift station located downstream of Butler Avenue.

b. Between Central Avenue and Wentworth Avenue outfall lines, construct 2,710 feet of 21" sewer parallel with the existing interceptor.

c. Between Wentworth Avenue and Grand Avenue outfall lines, construct the following lines parallel to the existing interceptor: 1,360 feet of 10", 250 feet of 15" and 150 feet of 18" sewers parallel with certain portions of the existing interceptor.



SCHEME "A"
COMBINED
SEWAGE TREATMENT FACILITIES
(TYPICAL)
(PONDING 4 Q SA)
SOUTH ST. PAUL, MINNESOTA 1969 Plate 23

Figure 23.

Flow to be stored to pass through a grit removal basin to remove 100% of 150 mesh and larger grit sized at 14,000 gallons/SF/day. Solids settled to be removed with an overhead traveling bridge hoist with clam shell. Fine settled material to be pumped to a cyclone separator with effluent to the sanitary sewer. Solids recovered to be hauled to a sanitary landfill. Solids contained in the cyclone effluent to be discharged to the sanitary plant for removal with other sanitary solids.

3. Scheme B

In this scheme the combined sewage up to a maximum of two and one-half ($2\frac{1}{2}$) times the average daily flow will be routed directly to the plant for treatment. Flows between two and one-half ($2\frac{1}{2}$) times the average daily flow and the maximum peak for a 10 year rainfall intensity will be routed to the pond for temporary storage. Flows greater than the maximum peak for a 10 year rainfall intensity will be routed to the river after chlorination in the detention pond.

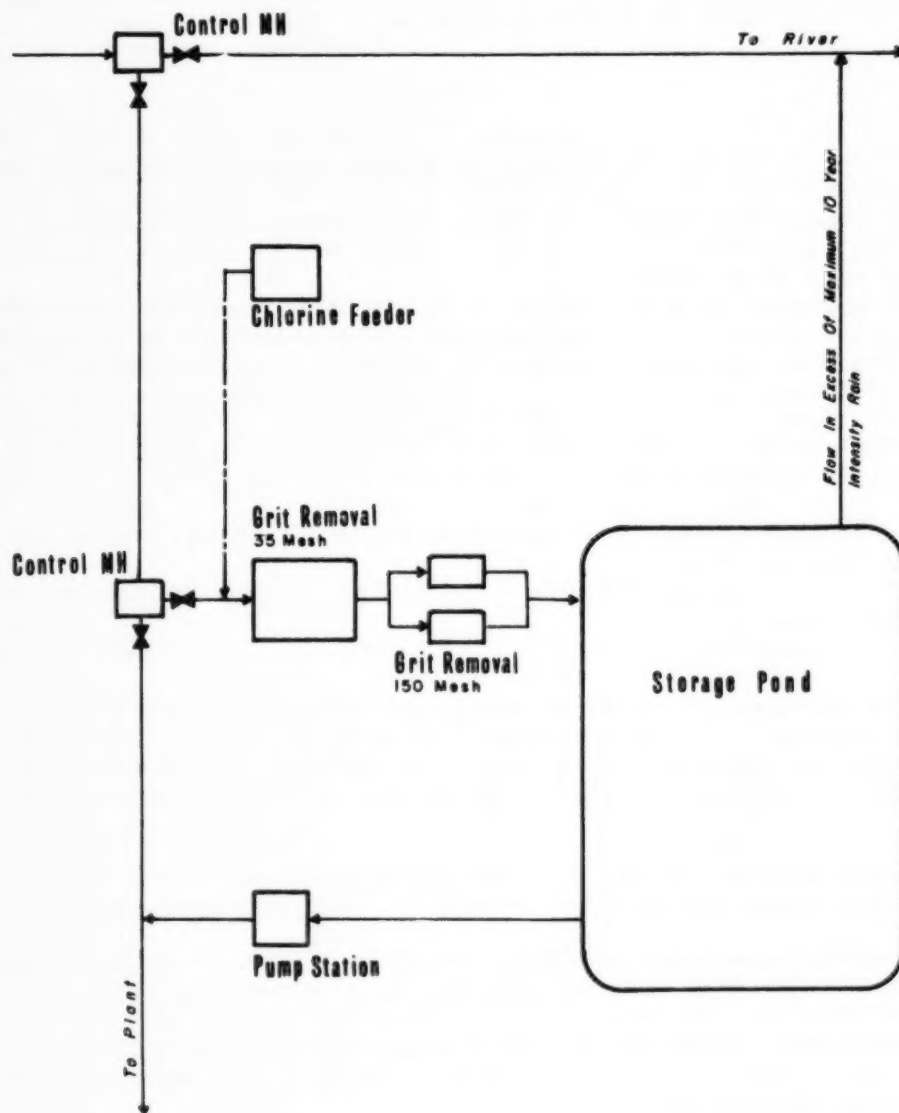
Controls will be similar to Scheme A except for the amounts routed to the plant, pond or river.

Ponds are sized to contain a 100-yr. volumetric rainstorm (6" of precipitation in 24 hours), following a 24-hour constant rate flow of 10 times the average daily flow during which time the rate of pumping equals the plant hydraulic capacity.

The combined sewage lift in the pond after a rainstorm will be pumped to the plant for treatment during the evening hours and week-ends. The combined rates of pumping from the ponds will be controlled so as not to exceed the hydraulic capacity of the plant.

Plate 24 shows a schematic layout of this proposed combined sewage pond system.

The following table indicates the basic data required for ponding of combined sewage under Scheme B for the drainage system under consideration:



SCHEME "B"
COMBINED
SEWAGE TREATMENT FACILITIES
(TYPICAL)
(PONDING 24HR-100YEAR STORM)
 SOUTH ST. PAUL, MINNESOTA - 1969

Figure 24.

System

	<u>Stanley Ave.</u>	<u>Central Ave. & Simons Ravine</u>	<u>Sixth Street</u>	<u>Malcby Street</u>
2.5 x Avg. Daily Flow cfs.	0.780	4.585	1.897	6.914
10.0 x Avg. Daily Flow cfs.	3.120	18.340	7.560	27.660
Max. Flow with 10-yr. Rain- fall Intensity cfs.	114.63	358.39	202.09	772.85
Pond Volume - Ac-Ft.	25.24	69.10	42.90	128.67
Pumping Capacity Max. GPM	670	1930	1200	5550
Chlorination Capacity Lbs/Day	800	2500	1400	5650
Grit Removal Chamber Sq.Ft.	6170	19150	10810	41600
Time Required to empty Pond Hrs.	228	228	228	228
Line Size to Pond In-Dia.	60	90	66	120
Line Size Pond Overflow In-Dia.	60	90	66	120
Chlorination Pond Size Length-ft.	513	713	748	865

It will be necessary to construct additional interceptor line capacity and increase existing lift station capacity in order to pass the flows during periods when the ponds are being emptied by pumping. The following additions to the present interceptor system will be required under this scheme.

- a. Increase pumping capacity from 350 gpm to 1,000 gpm in the existing lift station downstream of Butler Avenue.
- b. Between Central Avenue and Wentworth Avenue outfall lines, construct 2,710 feet of 18" sewer parallel with the existing interceptor.
- c. Between Wentworth Avenue and Grand Avenue outfall lines, construct 250 feet of 10" and 150 feet of 18" sewers parallel with certain portions of the existing interceptor.

Grit removal unit recommended would be sized to remove 100% of 150 mesh material. Facilities proposed would be two units, the first sized to remove 100% of 35 mesh grit and sized at 80,000 gallons/S.F./day. Grit would be removed from the basin with overhead hoist with a clam shell. The second or subsequent unit would be sized for 100% removal of 150 mesh materials. Basins would be sized at 14,000 gallons/S.F./day and be equipped with traveling bridge collectors with bottom scraper and water jets to remove settled material. Settled material to be pumped to a cyclone separator with effluent to sanitary sewer. Grit recovered in either or both units to be used as fill material or land filled as organic content dictates.

Storage basins would be sized to contain all waters until disposal. Storage basins should be drained to a sump for return of settled material to the grit recovery units. Provisions to be made either to allow flushing of settled material to the sump or to allow operation of dragline or front-end loader to remove settled material.

4. Scheme C

In this scheme, the combined flow up to a maximum of two and one-half ($2\frac{1}{2}$) times the average daily flow will be routed directly to the treatment plant. Flows exceeding two and one-half ($2\frac{1}{2}$) times the average daily flow will be chlorinated and routed to the river.

Chlorination before routing to the river will be done by injecting chlorine in the outfall line ahead of the chlorine detention pond. The chlorination ponds are sized to allow for fifteen (15) minutes detention time before the sewage is allowed to pass to the river. The same size of chlorination detention ponds will be used in Scheme A for each system. In Scheme B, flows exceeding the maximum peak for a 10-year rainfall intensity are to be chlorinated in the detention pond with overflow to the river.

Plate 25 shows a typical chlorination detention pond.

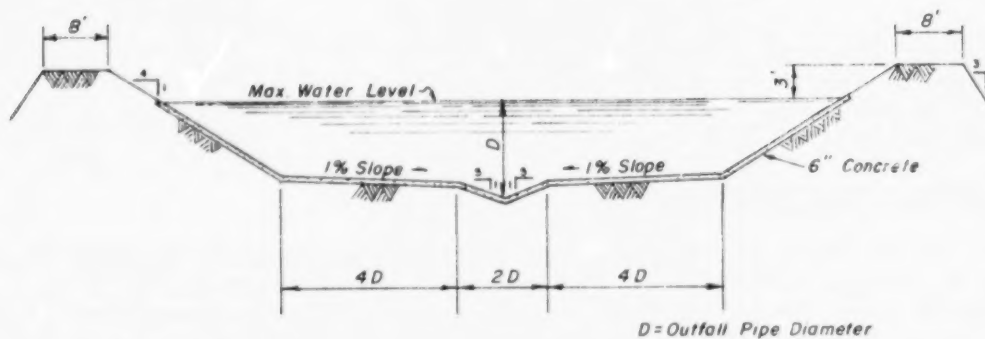
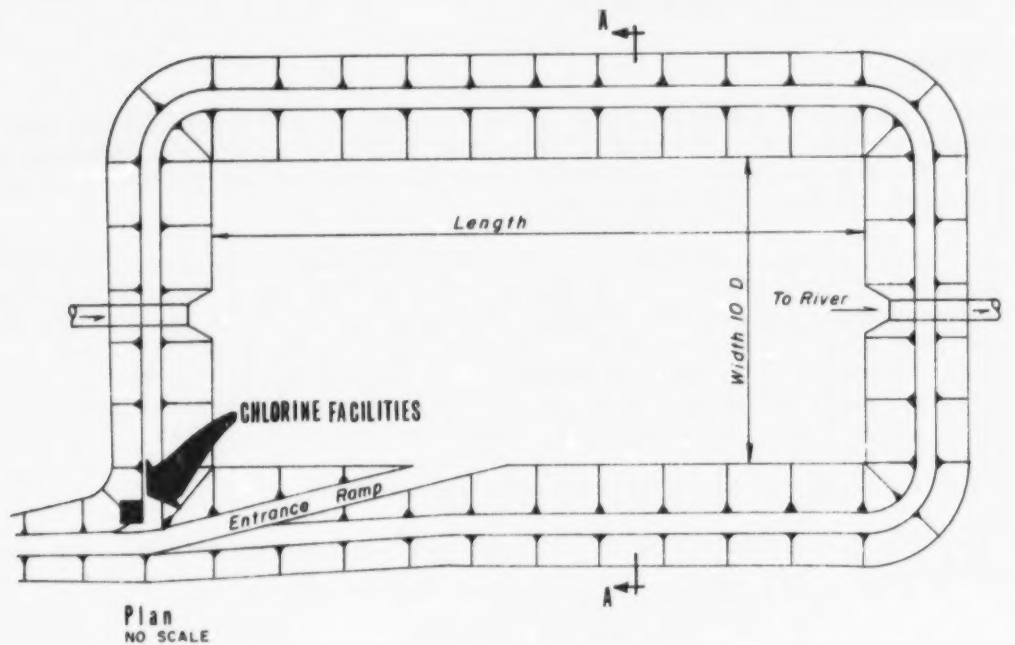
The following Table indicates the size of chlorination detention ponds for each system.

	Bottom Width Ft.	Length Ft.
Stanley Avenue	50	513
Central Avenue	75	713
Sixth Street	55	748
Maltby Street	100	865

EXISTING SEWAGE TREATMENT PLANT

The City of South St. Paul has a sanitary sewage treatment facility sized for an average daily flow of 17.5 MGD, with maximum capacity of 25 MGD. Plate 26 shows the anticipated hydraulic loading to the treatment facility during an average weekday, while industry is in operation, and a typical Saturday or Sunday.

As one means of treatment of combined flows intercepted and stored in retention ponds, it is suggested the night time and week-end hydraulic capacity of the existing sanitary treatment facility be utilized to receive and treat combined sewage. It has been calculated that approximately 15.4 MGD can be discharged to the sanitary plant on Saturday and Sunday, to load the plant to a 25 MGD hydraulic capacity. It has also been calculated that the treatment plant could be loaded to 25 MGD during week days from the period of approximately 10:00 P.M. to 8:00 A.M. during off peak hydraulic and organic loads to the facility. Approximately 5 MGD can be discharged to the sanitary plant on a given week day. Total weekly sanitary treatment facility capability utilized would then equal approximately 55 MG/week.



SCHEME "C"
COMBINED
SEWER CHLORINE DETENTION
POND
(TYPICAL)

SOUTH ST. PAUL, MINNESOTA

1969

Figure 25.

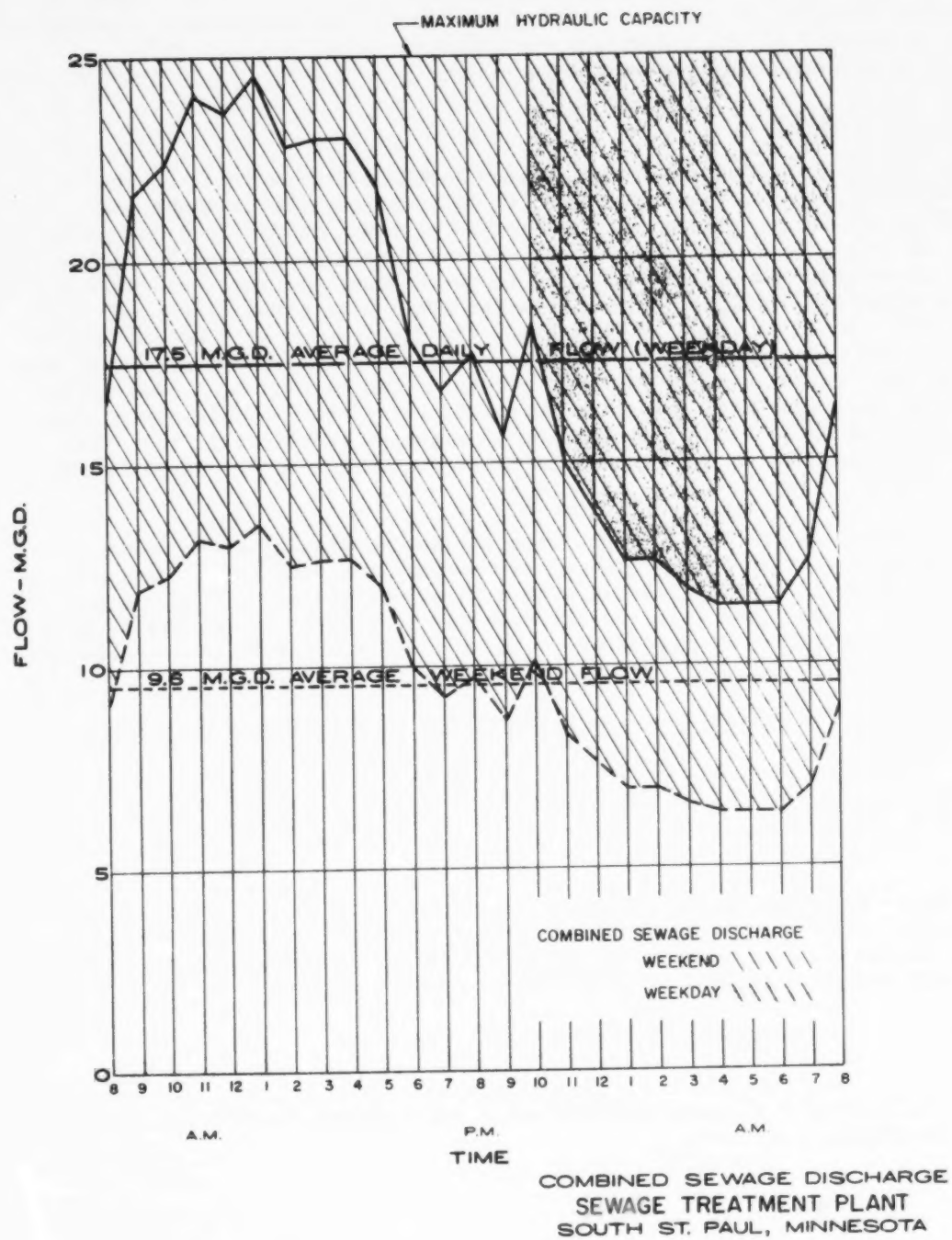


Figure 26.

It is anticipated that pond storage of combined sewage will effect a 35% reduction in BOD and a 65% reduction in suspended solids. Combined sewage received by the City Sanitary Treatment Facility should have BOD and suspended solids reductions approximately equal to that given sanitary sewage received by the facility.

1. Chlorination

The Minnesota Pollution Control Agency (MPCA) criteria that the total coliform group organisms be limited to 5,000 most probable number per 100 milliliters when applied to combined sewage is a difficult parameter to satisfy. It is estimated that an average contribution of 200 million coliform/person/day is contributed to the system. In raw form, this is approximately 52,000,000/100 millimeter MPN. A dilution factor of 10,000:1 would be required to satisfy the requirement of 5,000/100 millimeter MPN. The problem of effective bacterial reduction of discharge from combined systems can be solved by either of two methods or a combination of the two. The most effective method is to separate completely the raw sewage and collect it for treatment at sanitary facilities. The other extreme would be to chlorinate all the sewage prior to discharge to the receiving stream.

With regard to the required chlorine dosages to destroy effectively coliform bacterial in raw sewage, experiments indicate that a dose of 20 to 30 mg/liter is effective with a 15 minute contact period. Experiments also show that required chlorine dose decreases substantially with increased dilution of the sewage. The following information is presented regarding dilution and dosage of chlorine in mg/liter.

(2)

<u>Dilution in Dry Weather Flow</u>	<u>Chlorine Dose - mg/liter</u>
10	8.5
25	2.3
50	1.6
100	1.3

As previously stated, a contact period of approximately 15 minutes is required with chlorine dosage made proportional to flow and dilution and chlorine residual measured after the contact period to establish dosage and substantiate disinfection of combined sewage.

Table VI is a Frequency of Runoff tabulation sheet.

Table VII is a tabulation of dry weather and combined flows in multiples of dilution with chlorine dosages noted and chlorine required in terms of pounds/day feed rate.

Table No. VI

Column No.	Frequency of Runoff								
	<u>1</u>	<u>2</u>	<u>3</u>	<u>4</u>	<u>5</u>	<u>6</u>	<u>7</u>	<u>8</u>	<u>9</u>
	<u>A</u>	<u>C x A</u>	<u>Qsa</u> 250% DWF cfs	<u>3 Qsa</u>	Storm Flow @ intensity of 0.01"/ hour cfs	Storm Intensity to Produce 3 Qsa Inches/Hr. $(C \times A \times 0.01 \text{"/hr}) (\frac{3 \text{ Qsa}}{C \times A})$	of Time Rainfall in Excess of 3 Qsa Runoff (From Fig 7)(3.4-Col 7)	of Time Rainfall Less than 3 Qsa Runoff 2.0	% Reduction in Combined Discharge to River Intercepting 3 Qsa (Column 8 ÷ 3.4)
Stanley Ave.	99.6	50.5	0.78	2.34	0.50	0.047	1.4	2.0	59
Simons Ravine									
Central Ave.	252	138.2	4.58	13.74	1.38	0.099	0.75	2.65	78
Wentworth Ave.	674	356.0	4.92	14.76	3.56	0.041	1.7	1.7	50
Grand Ave.	188	107.6	2.99	8.97	1.08	0.083	0.85	2.55	75
6th Street	145	85.8	1.90	5.70	0.86	0.066	1.0	2.40	70
Maltby Street	720	397.3	6.91	20.73	3.97	0.052	1.2	2.2	65

A = Area in Acres

C = Coefficient of Runoff

DWF = Dry Weather Flow

cfs = Cubic Feet per Second

Qsa = 250% x DWF (cfs)

3 Qsa = Storm Flow

= 250 Gal/Capacity/Day

= 750 Gal/Capacity/Day

Combined Flow

= 1000 Gal/Capacity/Day = 4 Qsa

Table VII
Disinfection

	DWF cfs	Cl. @ #/day rate @ 25 mg/l	10 DWF Rate @ 8.5 mg/l	25 DWF cfs	Cl. @ 25 DWF @ 2.3 mg/l	50 DWF cfs	Cl. @ 50 DWF @ 1.6 mg/l	100 DWF cfs	Cl. @ 100 DWF @ 1.3 mg/l	10 yr. Max. Flow #/day	10 yr. Max. Flow #/day
1. Stanley Ave.	0.31	43	3.1	7.75	96	15.5	133	37.0	216	114	800
2. Central Ave.	1.83	252	18.34	46	570	92	790	183	1280	353	2470
3. Wentworth Ave.	1.97	270	19.69	49	608	98	845	197	1380	532	3720
4. Grand Ave.	1.19	165	11.97	30	372	60	515	119	834	262	1830
5. Sixth Street	0.75	105	7.56	19	236	37.5	322	75	525	200	1400
6. Maltby Ave.	2.76	380	27.66	69	855	138	1190	276	1930	765	5650

DWF = Dry Weather Flow
cfs = Cubic Feet per Second
Cl = Chlorine